

Guidelines for Determining Design Basis Ground Motions

Volume 2: Appendices for Ground Motion Estimation

Prepared by
ELECTRIC POWER RESEARCH INSTITUTE
in cooperation with the Joint Contractors
(Southern Electric International, Commonwealth Research Corporation,
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the Nuclear Management and Resources Council,
the U.S. Department of Energy and Sandia National Laboratories.

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Procedures currently used to assess the nature of earthquake ground motion in Eastern North America introduce considerable uncertainty to the design parameters of nuclear power plants and other critical facilities. This report examines that issue in-depth and provides an engineering model and guideline for selecting a site and assessing its seismic suitability.

INTEREST CATEGORIES

Nuclear seismic risk, design,
and qualification
Advanced light water
reactors
Risk analysis; management
and assessment
Nuclear plant life extension

KEYWORDS

Earthquakes
Site characterization
Hazard assessment
Seismic engineering
Seismology
Geotechnical engineering

BACKGROUND Eastern North America has sparse earthquake activity with rare occurrences of large earthquakes; thus, little data exists to empirically quantify the characteristics of ground motions. Procedures currently used to estimate ground motion effects in this region introduce considerable uncertainty into the process of developing seismic designs, either due to the procedure's subjectivity or the lack of physical calibration.

OBJECTIVES To develop generic relations for estimating ground motion appropriate for site screening; to develop a guideline for conducting a thorough site investigation needed to define the seismic design basis.

APPROACH The project team specifically considered ground motions resulting from earthquakes with magnitudes from 5 to 8, fault distances from 0 to 500 km, and frequencies from 1 to 35 Hz. To develop generic ground motion relations for Eastern North America, they used theoretical models calibrated against data from earthquakes throughout North America and the world. In these models, the contributions to ground motion, including its variability, were evaluated using physical representations of earthquake processes. Earthquake processes involve the initial generation of seismic energy or waves at the earthquake fault ("source effects"), followed by the propagation of seismic waves through the earth's crust ("path effects"), and finally the modification of seismic waves as they travel through soils near the earth's surface ("site effects"). The team also collected and analyzed extensive geotechnical data at three reference sites. This information provided the basis for developing a guideline to help assess site suitability.

RESULTS This project resulted in an engineering model for estimating earthquake ground motions in Eastern North America. The model considers a wide range of earthquake sizes and site conditions and may be used directly for site screening purposes. The work also resulted in a guideline for conducting geotechnical and seismic engineering investigations needed to determine the design basis for a site. This guideline is appropriate for investigating a wide range of site conditions and soil depths within and outside Eastern North America.

EPRI PERSPECTIVE Cost-effective seismic regulation of nuclear power plants requires site-specific definition of seismic ground motions. The development of engineering procedures for estimating earthquake ground motion can thus benefit both operating and future plants. For licensing application, these procedures are needed

to define the safe shutdown earthquake (SSE). The regulatory guidance found in Section 2.5 of the Standard Review Plan (NUREG 0800) is quite limited in scope and does not reflect the current state of knowledge on earthquake phenomena. With no accepted generic procedures in place, utilities constantly face uncertainty associated with site-specific developments and applications. These factors result in seismic design bases that are excessively conservative and/or contribute to licensing delays, regulatory instability, and high utility costs in the licensing process.

In 1988, EPRI completed a seismic hazard model for the central and eastern United States (NP-4726), including a ground motion model (NP-6074). The present work directly complements NP-4726, while replacing and going significantly beyond the results of NP-6074. The engineering ground-motion model can be used for screening potential sites before conducting extensive site investigations. The guideline provides needed background information to conduct an appropriate geotechnical and seismic engineering investigation of a site for licensing purposes. Additional EPRI reports that provide a basis for the current report include: NP-5577, NP-5875, NP-6304, TR-100409, TR-100410, TR-102261, and TR-102262.

This report is presented in five volumes. Essential background, approach and results are given mainly in Volume 1. Volumes 2, 3, and 4 are appendices containing detailed analyses. Volume 5 (licensed material) contains *Quantification of Seismic Source Effects*, which is summarized in Volume 1, Section 4.

PROJECT

RP3302

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ABSTRACT

This report develops and applies a method for estimating strong earthquake ground motion. The emphasis of this study is on ground motion estimation in Eastern North America (east of the Rocky Mountains), with particular emphasis on the Eastern United States and southeastern Canada. Specifically considered are ground motions resulting from earthquakes with magnitudes from 5 to 8, fault distances from 0 to 500 km, and frequencies from 1 to 35 Hz. The two main objectives were: (1) to develop generic relations for estimating ground motion appropriate for site screening; and (2) to develop a guideline for conducting a thorough site investigation needed to define the seismic design basis. For the first objective, an engineering model was developed to predict the expected ground motion on rock sites, with an additional set of amplification factors to account for the response of the soil column over rock at soil sites. The results incorporate best estimates of ground motion as well as the randomness and uncertainty associated with those estimates. For the second objective, guidelines were developed for gathering geotechnical information at a site and using this information in calculating site response. As a part of this development, an extensive set of geotechnical and seismic investigations was conducted at three reference sites. Together, the engineering model and guidelines provide the means to select and assess the seismic suitability of a site.

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EXECUTIVE SUMMARY

Introduction

This report develops and applies a method for estimating strong earthquake ground motion. The motivation for this development was the need for a systematic, physically based, empirically calibrated method that can be used to estimate ground motions for input to the design of nuclear power plants and other critical facilities. These ground motions are a function of the earthquake's magnitude and the physical properties of the earth through which the seismic waves travel from the earthquake fault to the site of interest. Procedures currently used to account for these effects introduce considerable uncertainty into the ground motion determination, either due to subjectivity of the procedure or the lack of physical calibration.

The emphasis of this study is on ground motion estimation in Eastern North America (east of the Rocky Mountains), with particular emphasis on the Eastern United States and southeastern Canada. Eastern North America is a stable continental region, having sparse earthquake activity with rare occurrences of large earthquakes. In the absence of large earthquakes within the region of interest, little data exist to empirically quantify the characteristics of ground motions associated with these events. While methods developed in more seismically active areas such as Western North America can be applied to Eastern North America, fundamental differences in the regional geology can lead to variations in ground motion characteristics. Therefore, empirically based approaches that are applicable for other regions, such as Western North America, do not appear to be appropriate for Eastern North America.

Recent advances in science and technology have now made it possible to combine theoretical and empirical methods to develop new procedures and models for estimating ground motion within Eastern North America. Specifically considered are ground motions resulting from earthquakes with magnitudes from 5 to 8, fault distances from 0 to 500 km, and frequencies from 1 to 35 Hz. The results of this report can be used to determine seismic hazards, provided the magnitudes and distances of potential earthquakes are predetermined. In particular, this report is intended for use in site screening as well as detailed characterization of ground motion at a site, such as may be required for structural design.

This study was conducted by a team of experts in seismology, geotechnical engineering, and seismic engineering. The investigations were carried out over a period of approximately 18 months from September 1991 to March 1993. Work included a series of focused workshops with project participants to help achieve consensus recommendations. The project was sponsored by the U. S. Department of Energy (DOE), Sandia National Laboratories, Southern Electric International, Commonwealth Research Corporation, Public Service Company of New Jersey, and the Electric Power Research Institute as part of the DOE's Early Site Permit Demonstration Program. The project was managed by the Electric Power Research Institute.

Objectives

There were two central objectives of the project: (1) to develop generic relations for estimating ground motion appropriate for site screening; and (2) to develop a guideline for conducting a thorough site investigation needed to define the seismic design basis. For the first objective, a set of relations was needed that could be used to predict the expected ground motion on rock or on soil for a future earthquake. The approach was to develop an engineering model consisting of relations appropriate for rock sites and an additional set of amplification factors to account for the response of the soil column over rock at soil sites. For the second objective, a guideline was developed for gathering geotechnical information at a site and using this information in calculating site response. Together, the engineering model and guideline provide the means to select and assess the seismic suitability of a site.

Approach

The method that was used to develop generic ground motion relations in this effort is markedly different from the approach of previous studies. In this study, theoretical models, which have been calibrated against data from earthquakes throughout North America and the world, are used to characterize earthquake ground motion in Eastern North America. In these models, the contributions to ground motion, including its variability, are evaluated using physical representations of earthquake processes. These processes involve the initial generation of seismic energy or waves at the earthquake fault ("source effects"), followed by the propagation of seismic waves through the earth's crust ("path effects"), and finally the modification of seismic waves as they travel through soils near the earth's surface ("site effects"). For a given earthquake magnitude and distance, the source, path, and site each contribute to the observed ground motion, as follows:

- The source controls both the seismic energy generated by rupture of an earthquake fault as well as the accompanying dynamic characteristics.
- The seismic path contributes to ground motion through reflection, refraction, and damping of seismic waves within the earth's crust in response to the various physical properties along the wave path.
- The site contributes to the evolution of seismic waves in much the same way as the path, though on a smaller scale. Site effects are a function primarily of soil depth and type.

The characteristics of the seismic source, path, and site effects form the basis for the parameters in the theoretical models.

The ground motion relations for rock sites were developed using a physically based, empirically calibrated ground motion model. In the model, a wide range of values was assigned to the ground motion parameters. Using the combination of all model parameters and their ranges of values, computer simulations produced hundreds of records of earthquake ground motion for each magnitude and distance considered. While each earthquake simulation represents a possible future earthquake, each earthquake is not equally likely to occur. Therefore, based upon extensive analyses of past earthquakes and comparisons to model predictions, distributions were assigned to the values for all model parameters. The parameter value distributions were based on partitioning their variability into two types: uncertainty, which is due to the lack of knowledge of earthquake characteristics; and randomness, which is due to the inherent variability of those characteristics. Finally, individual parameter weights were combined for each earthquake simulation to produce the appropriate "distribution" of earthquake ground motion for every magnitude, distance, and frequency considered. Together, these distributions constitute a family of functional relations that define the final engineering ground motion model for rock sites. In turn, the engineering model defines ground motion for median levels and associated variability.

To accommodate sites with soil overlying rock (referred to as local site effects), site amplification factors were developed for a range of soil types and depths representative of soil conditions in Eastern North America. The factors were derived by first accumulating data that describe the behavior of various soils during seismic loading. These

data were then used to assess the variability in seismic properties, especially the wave velocity as it changes with depth. In addition, seismic velocity and material damping data were gathered from three reference sites using a variety of field and laboratory techniques. The reference site data were used (1) to improve physical understanding of the dynamic processes of soil response and (2) to assess procedures for measuring the physical properties needed to estimate site effects. The estimation problem is particularly difficult because the seismic properties of soils change depending upon the level of shaking. The resulting “nonlinear” effects generally cause the ratio of soil-to-rock motions (i.e., soil amplification) to decrease as the corresponding rock motion increases. The quantification of these effects through theoretical modeling and comparisons to empirical data resulted in factors that describe the amplification of soils relative to rock for several soil categories. The amplification factors were developed for a wide range of rock motions and are given as median values with variability.

Finally, based upon extensive geotechnical data that were collected at the three reference sites and analyzed as part of this program, a guideline was developed for assessing soil characteristics and site response. This guideline applies to planning and conducting a systematic and thorough geotechnical investigation of soil properties at a potential site. Guidance is also provided for performing dynamic analyses required to determine the response of the soil column to earthquake shaking at (and beyond) the levels of motion of interest to the seismic design.

Conclusions

The engineering ground motion model developed in this study can be used for screening potential sites in Eastern North America before conducting extensive site investigations. However, the application of these procedures to site screening requires information regarding earthquake magnitudes and distances as well as certain site properties such as soil depth and site geology. Magnitudes and distances of potential earthquakes may be derived either probabilistically or deterministically.

The guideline—together with the results of investigations of the three reference sites—provides the means to conduct an appropriate geotechnical and seismic engineering site investigation. In all, this guideline is appropriate for use given a wide range of site conditions and soil depths. While there are certain soil types (e.g., those with liquefaction potential) for which this guideline may not be directly applicable, it may be used widely both within and outside Eastern North America.

The information compiled in this report represents a comprehensive assessment of the nature of earthquake ground motion in Eastern North America. The results incorporate best estimates of ground motion as well as the randomness and uncertainty associated with those estimates for a wide range of earthquake magnitudes, distances, and frequencies. Overall, the results of this study will be useful in performing seismic hazard evaluations and establishing seismic design standards for many years to come.

Organization

The results of this study are presented in five volumes. *Volume I: Methodology and Guidelines for Estimating Earthquake Ground Motion in Eastern North America*, representing the main body of the report, presents the model development and summarizes the key results and conclusions of the study. *Volume II: Appendices for Ground Motion Estimation*, presents the appendices to Sections 2 to 7 of Volume I, and consists primarily of data and details of analyses used to develop the engineering ground motion model and geotechnical guidelines. *Volume III: Appendices for Field Investigations*, and *Volume IV: Appendices for Laboratory Investigations*, present the details of field and laboratory investigations of reference sites; Section 8 of Volume 1 constitutes a summary of these appendices. *Volume V: Seismic Source Effects*, presents separately (as a licensed report) the analyses of the seismic source performed for input to the engineering ground motion model; a summary of this volume is given as Section 4 of Volume 1.

APPENDIX 2.A

EASTERN NORTH AMERICAN EMPIRICAL GROUND MOTION DATA

This appendix describes the data base of ground motion recordings from eastern North America (ENA) compiled for this project. The tectonic region of interest consists of the region extending from the east slope of the Rocky Mountains to the Atlantic coast.

The compiled data consist of all recordings obtained on strong motion instruments in eastern North America and seismographic network data from magnitude m_b 5.0 and greater earthquakes recorded at distances of 500 km or less. A few smaller events recorded by the Eastern Canadian Telemetered Network (ECTN) that were used to evaluate various numerical modeling studies were also included in the data base. The data bases of the National Center for Earthquake Engineering Research (NCEER) and the National Earthquake Information Center (NEIC) were searched to identify potential strong motion recordings. All identified strong motion records were then entered into a structured data base format that consists of four interrelated tables (Tables 2.A-1 through 2.A-4). The four tables are: the earthquake table (2.A-1) that contains information characterizing the earthquakes that produced the strong motion recordings, the site table (2.A-2) that contains information characterizing the recording stations at which the records were obtained, the record table (2.A-3) that contains the basic data describing the recordings, and the spectral table (2.A-4) that contains computed spectral ordinates for each record for frequencies above 1 Hz. In addition, brief descriptions of the site geology at each of the recording stations are presented in Table 2.A-5.

2.A.1 Earthquake Data

Table 2.A-1 lists the earthquakes for which ground motion recordings were collected. Each earthquake was given a unique identification code to enable correlation between tables. The data entered for each earthquake consists of the following parameters:

- Origin time: year, month, day, hour, minute, and second
- Location: Crustal structure zone, latitude, longitude, and focal depth
- Source properties: magnitude (M_s , m_b , m_{Lg} , M), seismic moment, reported stress drop (either Brune or RMS), and focal mechanism
- Comments on information
- References

The crustal structure zone location refers to the regionalization of the eastern North American crust discussed in Section 5.

When alternative estimates were available for various source parameters they have been entered on separate lines of Table 2.A-1. Because of the length of the information entered for each earthquake, Table 2.A-1 is broken into three parts, the first part contains the earthquake data, the second part contains any comments, and the third part contains the references. Within each part, the unique earthquake identification code is repeated on each line for cross referencing.

The ground motion data base consists of recordings from 51 earthquakes. Approximately half of these are aftershocks or secondary events in earthquake sequences.

2.A.2 Site Data

Table 2.A-2 lists the recording stations at which ground motion recordings from the earthquakes in Table 2.A-1 have been obtained. Each station is given a unique identification code to enable correlation between tables. The identification code consists of the codes assigned by the various agencies that maintain the stations; (the U.S. Geological Survey (USGS), the Geological Survey of Canada (GSC), the National Center for Earthquake Engineering Research (NCEER), or the individual network operators. Station codes for most stations are listed in Busby (1990), and Munro and Weichert (1989). The data entered for each earthquake consists of the following parameters:

- Station name and location: latitude, longitude, and crustal structure zone
- Station classification: instrument housing, local geology, site conditions
- Instrument type
- Comments

The station classification consists of a three-part letter code, which is explained in detail at the end of Table 2.A-2. The first code classifies the instrument housing. Instruments with letter codes I, A, and B are considered free field instruments, that is instruments that provide ground surface recordings expected to be free of significant building effects. The second code refers to the general site geology, providing a measure of the anticipated shear wave velocity gradient within the upper few kilometers of the crust. Letter codes A and B indicate sites underlain by hard rock where it is expected that there is only a small velocity gradient and thus little amplification of seismic waves as they approach the surface. Letter codes C and D refer to sites where a significant velocity gradient is anticipated within the upper few kilometers of the crust, leading to amplification of seismic waves (see discussion in Section 5.3). The third part classifies the near surface geology into generalized site categories. In general, the information on subsurface conditions beneath the stations is very limited and the detailed site categories developed in Section 6 could not be used. Instead, sites were classified as either rock, shallow soil (<20 m), intermediate soil (20–100 m), or deep soil (>100 m) based on the available information. The descriptions of the recording station site conditions upon which the site classification codes were based are presented in Table 2.A-5.

The geologic conditions listed in Table 2.A-5 were compiled from examination of large- to small-scale geologic maps (scales from 1:24,000 to 1:500,000 or smaller). The descriptions define local geologic conditions for each site, including significant changes in geologic conditions over lateral distances of approximately two to five kilometers from the station. The approximate thickness of stratigraphic units was evaluated from geologic maps or from descriptions of nearby borings or local stratigraphic conditions. The near-surface site conditions (within a few meters) are listed where this information was readily available from published reports. The information on the presence or absence, and the exact thickness of unconsolidated deposits beneath each station was not uniformly available. Rather, the descriptions indicate the maximum or average thickness of unconsolidated materials in the vicinity of each station.

A general description of the topography near each station follows the geologic descriptions. Additional information for some stations includes a description of the instrument setting. A confidence rating of the geologic conditions is also given for each station. This rating scheme was developed to indicate the level of uncertainty in the geologic conditions for a given site. The levels of confidence are defined as follows.

High—geology of site and vicinity well known (for thicknesses of 10s to 100s of meters and distances of several kilometers from the station).

Moderate—geology of area well known, site conditions (nature, extent, and thickness of underlying rocks) uncertain.

Low—geology of area and site uncertain; observed only from small-scale maps (i.e., 1:250,000 scale or smaller), or station located slightly off of available maps.

2.A.3 Recordings

Table 2.A-3 lists the ground motion recordings contained in the data base. Each recording is identified by the earthquake code from Table 2.A-1 and the station code from Table 2.A-2. The data for each recording consists of the following:

- Source-to-site distance: hypocentral, epicentral, and closest (fault) distance
- Components
- Record processing parameters: high pass and low pass filters, and digitization time step
- Peak motions for each component
- Source of corrected record
- Response spectral ordinates (Table 2.A-4)

The hypocentral and epicentral distances were computed from the earthquake and station location parameters listed in Tables 2.A-1 and 2.A-2. Closest distance to rupture surface was available only for the December 23, 1985 Nahani earthquake and was based on a fault plane inferred from the aftershock distribution.

The collected accelerograms were processed according to the standard practices used by the various agencies. The data from NCEER was either processed by the U.S. Geological Survey or by NCEER using the standard U.S. Geological Survey processing methodology. Data from the ECTN network were processed following the procedure outlined in Atkinson and Mereu (1992), which consists of removal of instrument response, band-pass filtering the records within the dynamic frequency range of the ECTN instruments, and conversion to acceleration in the frequency domain. The records supplied by Pacific Engineering and Analysis (PE&A) were instrument corrected and band-pass filtered within the frequency range of high signal-to-noise ratio. The high pass and low pass filter frequencies applied to each record are listed in Table 2.A-3. These values indicate the useful frequency range for each record.

Response spectral amplitudes (5 percent damping) were computed for each of the recordings for 14 frequencies in the range of 1 to 100 Hz. The computed spectral ordinates are listed in Table 2.A-4. Values of spectral accelerations are listed only for frequencies that lie in the frequency band defined by the filter parameters used in the record processing (see Table 2.A-3).

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Table 2.A-1

Earthquake Data For ENA Ground Motion Data Base

Rec	EQID	EQNAME	DATE	HR	MIN	SEC	ZONE	LAT	LONG	DEPTH	Ms	mb	MLg	M	MD	MU	MO	EXP	STDRP	FP1S	FP1D	FP1R	FP2S	FP2D	FP2R	FAULT
1	NW750600	NEW MADRID, MO	1975.06.13	22	40	27.00	H	36.530	-89.660	9.00	4.2	4.2	3.7				4.6	21		N85E	60S		N5E	70 W		
2	NW760300	NEW MADRID, MO	1976.03.25	0	41	20.50	H	35.600	-90.500	12.00	5.0	5.0	4.6				9.8	22		N323E	63NE		N40E	65NW		FP2
3	NW760301	NEW MADRID, MO	1976.03.25	1	0	11.90	H	35.600	-90.500	16.00	4.5	4.5	4.2				2.5	22		N45E	80SE		N309E	60SE		FP1
4	JK780800	MONTICELLO, SC	1978.08.27	10	23		C	34.308	-81.334	0.10			2.6				1.0	20	44							
5	JK780800	MONTICELLO, SC	1978.08.27	10	23		C						2.9				2.7	20	8							
6	JK781000	MONTICELLO, SC	1978.10.27	7	26		C	34.299	-81.338	0.20			2.9				8.7	19	304							
7	JK781000	MONTICELLO, SC	1978.10.27	7	26		C						2.9				2.6	20	13							
8	JK781001	MONTICELLO, SC	1978.10.27	16	27		C	34.003	-81.328	0.20			2.9				3.1	20	13							
9	JK791000	MONTICELLO, SC	1979.10.07	0	0		C						2.8													
10	JK791001	MONTICELLO, SC	1979.10.16	7	6		C	34.310	-81.335	0.07			3.0				3.5	20	108							
11	JK791001	MONTICELLO, SC	1979.10.16	7	6		C						2.9				3.2	20	77							
12	NB820100	NEW BRUNSWICK (A5)	1982.01.11	21	41	8.00	B	46.980	-66.670	6.00	5.4	5.5	5.2													
13	NB820100	NEW BRUNSWICK (A5)	1982.01.11	21	41	8.00	B					5.1	5.0													
14	NB820101	NEW BRUNSWICK (A2)	1982.01.17	12	24		B	46.980	-66.620	2.00			2.7	2.0			1.4	19	44							
15	NB820101	NEW BRUNSWICK (A2)	1982.01.17	12	24		B					2.4					4.9	19	33							
16	NB820102	NEW BRUNSWICK (A3)	1982.01.17	13	32	59.89	B	46.990	-66.630				2.8	2.1			2.0	19	43							
17	NB820102	NEW BRUNSWICK (A3)	1982.01.17	13	32	59.89	B					2.1					1.9	19	13							
18	NB820102	NEW BRUNSWICK (A4)	1982.01.17	13	32	59.89	B	46.980	-66.610	5.50			2.2				2.5	19	13							
19	NB820103	NEW BRUNSWICK (A4)	1982.01.17	13	33	55.71	B					3.5	3.0				4.0	20	270							
20	NB820103	NEW BRUNSWICK (A4)	1982.01.17	13	33	55.71	B					3.1					5.6	20	23							
21	NB820103	NEW BRUNSWICK (A4)	1982.01.17	13	33	55.71	B					2.9					3.3	20								
22	NB820103	NEW BRUNSWICK (A4)	1982.01.17	13	33	55.71	B					3.6	3.2				6.1	19	72							
23	NB820104	NEW BRUNSWICK (A7)	1982.01.18	19	34	49.17	B	47.010	-66.610				3.0	2.5			9.0	19	110							
24	NB820104	NEW BRUNSWICK (A7)	1982.01.18	19	34	49.17	B					2.6					1.5	20								
25	NB820104	NEW BRUNSWICK (A7)	1982.01.18	19	34	49.17	B					2.7					9.4	18								
26	FK820100	FRANKLIN FALLS, NH	1982.01.19	0	14	42.00	B	43.494	-71.593	3.40	4.6	4.8	4.3	4.7	4.4	3.0	22									
27	NB820105	NEW BRUNSWICK (A7.5)	1982.01.19	11	59		B	46.990	-66.620			2.5	1.9				9.4	18								
28	NB820105	NEW BRUNSWICK (A7.5)	1982.01.19	11	59		B					2.8	2.5				1.3	19	21							
29	NB820106	NEW BRUNSWICK (A8)	1982.01.20	10	0		B	47.050	-66.610				2.0				7.4	19	15							
30	NB820106	NEW BRUNSWICK (A8)	1982.01.20	10	0		B					2.5					7.4	19	120							
31	NB820106	NEW BRUNSWICK (A8)	1982.01.20	10	0		B					2.3					4.0	19	13							
32	NB820107	NEW BRUNSWICK (A12)	1982.01.21	0	39		B	46.980	-66.600			2.5	2.1				1.9	19	30							
33	NB820107	NEW BRUNSWICK (A12)	1982.01.21	0	39		B					2.3					3.0	19	35							
34	NB820107	NEW BRUNSWICK (A12)	1982.01.21	0	39		B					2.0					1.2	19	150							
35	NB820107	NEW BRUNSWICK (A12.5)	1982.01.21	0	39	55.72	B	46.980	-66.600	6.00			2.2				2.7	19								
36	NB820108	NEW BRUNSWICK (A12.5)	1982.01.31	11	23		B					2.4														
37	NB820300	NEW BRUNSWICK (A13)	1982.03.31	21	2	20.40	B	47.000	-66.570	4.00	5.0	4.8	4.0	4.5												
38	NB820300	NEW BRUNSWICK (A13)	1982.03.31	21	2	20.40	B					4.2														
39	NB820400	NEW BRUNSWICK (A14)	1982.04.02	13	50		B	47.000	-66.570			4.3														
40	NB820401	NEW BRUNSWICK (A15)	1982.04.11	18	0		B	47.000	-66.570			4.1														
41	NB820402	NEW BRUNSWICK (A16)	1982.04.28	6	36		B	47.000	-66.600			3.4														
42	NB820500	NEW BRUNSWICK (A17)	1982.05.06	16	28	7.00	B	47.000	-66.600			4.0	3.4													
43	NB820600	NEW BRUNSWICK (A19)	1982.06.16	11	43		B	47.010	-66.950	8.00	4.7	4.6	4.0	3.8												
44	NB820700	NEW BRUNSWICK (A18)	1982.07.28	5	35		B	47.000	-66.600			3.7	3.3													

Table 2.A-1 (continued)
Earthquake Data For ENA Ground Motion Data Base

Rec EQID	EQNAME	DATE	HR	MIN	SEC	ZONE	LAT	LON	DEPTH	Ms	mb	MLg	M	MD	MU	MO	EXP	STDRP	FP1S	FP1D	FP1R	FP2S	FP2D	FP2R	FAULT
45 GN831000	GOODNOW, NY	1983.10.07	10	18	46.00	J	43.938	-74.258	7.50	5.1	5.1	4.8	4.8			1.9	23		180	61	81				
46 GN831000		1983.10.07	10	18	46.00	J					5.6	4.9	5.3												
47 GN831000		1983.10.07	0	0	0								5.1												
48 GN831000		1983.10.07	0	0	0								4.9												
49 NH851100	NAHANT, CAN (F1)	1985.11.09	4	46		J	62.210	-124.270	18.00	4.8	4.8	4.6	4.6			1.5	26	50	N175E	25W	90	N355E	65E	90	FP1
50 NH851200	NAHANT, CAN	1985.12.23	5	16	6.00	J	62.187	-124.263	6.00	6.9	6.4	6.7	6.7			1.8	26	54	N172E	22W	116				FP1
51 NH851200		1985.12.23	5	16	6.00	J			6.20			6.8	6.8												FP1
52 NH851200		1985.12.23	5	16	6.00	J						6.7	6.7			1.2	26		N186E	45W	121				FP1
53 NH851201	NAHANT, CAN (A1)	1985.12.25	15	42		J	62.020	-124.130	6.00	5.7	5.7	5.0	5.3			1.0	23		N0E	76W	171				FP1
54 NO860100	NORTHEASTERN OHIO	1986.01.31	16	46	42.30	J	41.646	-81.158	5.50	4.9	5.0	4.6	5.0						N55E	73	171				
55 NO860100		1986.01.31	16	46	42.30	J						5.3	5.3						N22E	81	160				
56 NO860100		1986.01.31	16	46	42.30	J						5.3	5.3						N21E	84	161				
57 A871111	EASTERN CANADA	1987.11.11	0	0		J	45.776	-75.336	21.10			3.5	3.0												
58 A880310	EASTERN CANADA	1988.03.10	0	0		J	46.340	-75.670	12.00			3.7	3.7												
59 MS880800	MASSENA, NY	1988.08.09	13	57	26.90	J	45.000	-74.983	9.50	3.5	3.5	3.1	3.1	3.4											
60 MS880800		1988.08.09	13	57	26.90	J						3.4	2.8												
61 SG881100	SAGUENAY, CAN (F1)	1988.11.23	9	11	27.30	J	48.134	-71.203	28.00	4.2	4.4	4.8	4.5			8.0	24	28	N325E	67E	54	N208E	41W		
62 SG881101	SAGUENAY, CAN	1988.11.25	23	46	4.50	J	48.117	-71.184	29.00	5.8	5.9	6.5	5.9			6.9	24								
63 SG881101		1988.11.25	23	46	4.50	J						5.8	5.8			8.0	24								
64 SG881101		1988.11.25	23	46	4.50	J						5.9	5.9			8.0	24								
65 SG881101		1988.11.25	23	46	4.50	J			26.00			5.7	5.7			5.0	24	160	N320E	65E	78				
66 NM881200	NEW MADRID, MO	1988.12.31	14	24	20.50	H	36.185	-89.428	5.00	3.5	3.5	3.5	3.5												
67 NM890400	NEW MADRID, MO	1989.04.27	16	47	49.80	H	36.006	-89.768	10.00	4.6	4.7	4.7	4.7	4.3											
68 MS890700	MASSENA, NY	1989.07.19	11	58	55.90	J	45.053	-74.902	9.00	1.9	1.9	1.9	1.9												
69 A900421	EASTERN CANADA	1990.04.21	0	0		J	47.553	-70.070	9.60			3.1	2.8												
70 CG900900	CAPE GIRARDEAU	1990.09.26	13	18	51.30	H	37.165	-89.577	12.00	4.7	5.0	4.7	4.8												
71 ML901000	MOUNT-LAURIER QUEBEC	1990.10.19	7	1	57.40	J	46.470	-75.590	13.00	4.2	4.6	5.1	5.0												
72 NM901100	NEW MADRID REGION	1990.11.09	3	39	15.10	H	36.540	-89.620	9.00			3.6	3.4												
73 SQ901200	SOUTHERN QUEBEC	1990.12.31	3	53	58.30	J	47.579	-72.556	18.00	4.2															
74 NM910500	NEW MADRID (FS), MO	1991.05.04	1	13	59.12	H	36.553	-89.799	7.10			2.2	2.1			2.0	19								
75 NM910501	NEW MADRID, MO	1991.05.04	1	18	54.60	H	36.555	-89.802				4.6	4.4			5.0	22								
76 NM910501		1991.05.04	1	18	54.60	H						4.1	4.1			2.0	22								

Table 2.A-1 (continued)

Earthquake Data For ENA Ground Motion Data Base

REC	EQID	COMMENT	REF1
1	NM750600	4-4.25 mb, lo VI, P/T axes from S-wave fmech (+/- 10 deg, ref2)	Herrmann et al (1977)
2	NM760300	lo VI, P/T axes from S-wave fmech (+/- 10 deg)	Herrmann and Canas (1978)
3	NM760301	P/T axes from S-wave fmech (+/- 10 deg)	Herrmann and Canas (1978)
4	JK780800	Mo, stress drop from 180 deg (accel) record	Fletcher (1982)
5	JK780800	Mo, stress drop from 90 deg (displ) record	Fletcher (1982)
6	JK781000	Mo, stress drop from 180 deg (accel) record	Fletcher (1982)
7	JK781000	Mo, stress drop from 90 deg (displ) record	Fletcher (1982)
8	JK781001	Silva's list. Same Mo, stress drop from 180 and 90 deg	Fletcher (1982)
9	JK791000		
10	JK791001	Mo, stress drop from 180 deg (accel) record	Fletcher (1982)
11	JK791001	Mo, stress drop from 90 deg (displ) record	Fletcher (1982)
12	NB820100	mb from USGS, M is average value (see ref2)	Choy et al (1983)
13	NB820100		Silva (personal communication)
14	NB820101	mLg values from ad hoc regression (see ref1)	Mueller and Cranswick (1985)
15	NB820101	Mo and stress drop from other station (ref1)	Mueller and Cranswick (1985)
16	NB820102	Mo and stress drop from other station (ref1)	Mueller and Cranswick (1985)
17	NB820102	add'l source info avail (ref1)	Mueller and Cranswick (1985)
18	NB820102		Saikia and Herrmann (1985)
19	NB820103		Mueller and Cranswick (1985)
20	NB820103	Mo and stress drop from other station (ref1)	Mueller and Cranswick (1985)
21	NB820103	add'l source info avail (ref1)	Saikia and Herrmann (1985)
22	NB820103		Atkinson (1993)
23	NB820104		Mueller and Cranswick (1985)
24	NB820104	Mo and stress drop from other station (ref1)	Mueller and Cranswick (1985)
25	NB820104	add'l source info avail (ref1)	Saikia and Herrmann (1985)
26	FK820100	Mu is an average M value (see ref3)	Brown and Ebel (1985)
27	NB820105		Mueller and Cranswick (1985)
28	NB820105	Mo and stress drop from other station (ref1)	Mueller and Cranswick (1985)
29	NB820106		Mueller and Cranswick (1985)
30	NB820106		Mueller and Cranswick (1985)
31	NB820106		Mueller and Cranswick (1985)
32	NB820107		Mueller and Cranswick (1985)
33	NB820107		Mueller and Cranswick (1985)
34	NB820107	add'l source info avail (ref1)	Mueller and Cranswick (1985)
35	NB820107		Saikia and Herrmann (1985)
36	NB820108		Silva (personal communication)
37	NB820300	dpth inferred (ref2), comp. fmechs (ref3), mu is mLg (WES)	Weichert et al (1982) [loc, mLg]
38	NB820300	M and Mu are both M values (see ref1)	Boore and Atkinson (1987)
39	NB820400		Weichert et al (1982)
40	NB820401		Weichert et al (1982)
41	NB820402		Weichert et al (1982)
42	NB820500		Weichert et al (1982)
43	NB820600	comp. fmechs (ref2), M and Mu are both M values (see ref3)	Weichert et al (1982)
44	NB820700		Weichert et al (1982)

Table 2.A-1 (continued)
Earthquake Data For ENA Ground Motion Data Base

Rec	EQID	COMMENT	REF1
45	GN831000	source duration = .45 sec (ref1)	Nabelek and Suarez (1989)
46	GN831000	M and Mu are both M values (see ref1)	Boore and Atkinson (1987)
47	GN831000	mLg and Mu are both mLg values (see ref1)	Boore and Atkinson (1987)
48	GN831000	M and Mu are both M values (see ref1)	Boore and Atkinson (1987)
49	NH851100		*** NOAA
50	NH851200	mB and Ms from NEIS	Wetmiller et al (1988)
51	NH851200	z+/-1.5km, Mo+/-4E26, sdt+/-14bar	Choy and Boatwright (1988)
52	NH851200		Barker (1989)
53	NH851201		* NCEER
54	NO860100	1st motion fmech, Mo=1-3E23, z=4-6, Io=VI-VII (ref2)	Glassblower and Borchardt (1990)
55	NO860100	M and Mu are both M (see ref2), S-wave inversion fmech	Nicholson et al (1988)
56	NO860100	Harvard solution fmech	Nicholson et al (1988)
57	A871111		Atkinson (1993)
58	A880310		Hough et al (1989)
59	MS880800	Mu is mLg from OTT (see ref1)	Atkinson (1993)
60	MS880800		North et al (1989)
61	SG881100	poor fmech control (ref1)	North et al (1989)
62	SG881101	stress drop not well constrained	North et al (1989)
63	SG881101	Harvard moment	North et al (1989)
64	SG881101		Carabajal and Barker (1991)
65	SG881101	stress drop +/- 2.3	Somerville et al (1990)
66	NH881200		** PDE
67	NH890400	Mu is mLg from NEIS	** PDE
68	MS890700	not in PDE	* NCEER
69	A900421		** PDE
70	CG900900	Mu is a mLg value (see ref1)	** PDE
71	ML901000	Mu is a mLg value (see ref2)	** PDE
72	NH901100		
73	SD901200		
74	NH910500		Friberg et al (1991)
75	NH910501	ref2 reports they have fmech, moment from far field spect.	Friberg et al (1991)
76	NH910501	moment calculated using impedance term, stress drop 87-130	Friberg et al (1991)

Table 2.A-1 (continued)
Earthquake Data For ENA Ground Motion Data Base

Rec	EQID	REF1	REF2	REF3
1	NM750600	Herrmann and Canas (1978) [P/T axes]		
2	NM760300	Herrmann (1977) [nodal planes]		
3	NM760301	Herrmann (1977) [nodal planes]		
4	JK780800			
5	JK780800			
6	JK781000			
7	JK781000			
8	JK781001			
9	JK791000			
10	JK791001			
11	JK791001			
12	NB820100	Boore and Atkinson (1987) [mLg, M]		
13	NB820100			
14	NB820101			
15	NB820101			
16	NB820102			
17	NB820102			
18	NB820102			
19	NB820103			
20	NB820103			
21	NB820103			
22	NB820103			
23	NB820104			
24	NB820104			
25	NB820104			
26	FK820100			
27	NB820105	Pulli et al (1983) [z, mb, Mo, focal mech]		Boore and Atkinson (1987) [mLg, Mu]
28	NB820105			
29	NB820106			
30	NB820106			
31	NB820106			
32	NB820107			
33	NB820107			
34	NB820107			
35	NB820107			
36	NB820108			
37	NB820300	Choy et al (1983)		Wetmiller et al (1984)
38	NB820300			
39	NB820400			
40	NB820401			
41	NB820402			
42	NB820500	Atkinson (1993) [mLg, M]		
43	NB820600	Wetmiller et al (1984)		Boore and Atkinson (1987)
44	NB820700	Atkinson (1993) [mLg, M]		

Table 2.A-1 (continued)
Earthquake Data For ENA Ground Motion Data Base

Rec	EQID	REF2	REF3
45	GN831000		
46	GN831000		
47	GN831000		
48	GN831000		
49	NH851100		
50	NH851200		
51	NH851200		
52	NH851200		
53	NH851201	Horner et al (1990) [Mu, fmech]	
54	NO860100	Nicholson et al (1988) [ml-g, focal mech]	** PDE [mb]
55	NO860100	Boore and Atkinson (1987) [ml-g, M, Mu]	
56	NO860100		
57	A871111		
58	A880310		
59	MS880800	** PDE [mLg]	* NCEER [mb, M]
60	MS880800		
61	SG881100	* NCEER [Ms, M]	** PDE [z, mb]
62	SG881101		
63	SG881101		
64	SG881101		
65	SG881101		
66	NH881200	* NCEER [mb, M]	
67	NH890400	* NCEER [mb, M]	
68	MS890700		
69	A900421	Atkinson (1993) [M]	
70	CG900900		
71	ML901000	Lamontagne (1991) [mu->mLg]	
72	NH901100		
73	Sq901200		
74	NW910500	* NCEER [loc, sec]	* NCEER [sec]
75	NW910501	Chiu et al (1991) [loc, z]	
76	NW910501		

Table 2.A-2

Recording Station Data For ENA Ground Motion Data Base

REC	ST. NO	DESP	ST. LAT	ST. LON	SZONE	CL_1	CL_2	CL_3	IS. TYPE	COMMENTS
1	PAL	PALISADES, N.Y	41.004	-73.909	J	I	A	A	SSA-1 Acc.	
2	NEWC	NEWCOMB, N.Y	43.971	-74.224	J	I	A	A	SSA-1 Acc.	
3	JORD	TUPPER LAKE, N.Y	44.378	-74.605	J	C	A	A	AC	
4	MSNA	MASSENA, N.Y	44.998	-74.847	J	I	B	A	SSA-1 Acc.	
5	LYON	LYON MOUNTAIN, N.Y	44.726	-73.911	J	I	A	A	SSA-1 Acc.	
6	EMME	EAST MACHIAS, MAINE	44.739	-67.489	B	I	A	A	SSA-1 Acc.	
7	MIME	MILLO, MAINE	45.244	-69.043	B	I	B	A	SSA-1 Acc.	
8	ISFL	ISLAND FALLS, MAINE	46.031	-68.206	B	I	A	A	SSA-1 Acc.	
9	DICKY	DICKEY, MAINE	47.101	-69.082	B	I	B	A	SSA-1 Acc.	
10	GSC1	GSC SITE 1 - St-FERREOL, QUE	47.126	-70.828	J	I	A	A	SMA-1 Acc.	
11	GSC2	GSC SITE 2 - QUEBEC, QUE	46.778	-70.275	J	C	B	A	SMA-1 Acc.	
12	GSC5	GSC SITE 5 - TADOUSSAC, QUE	48.143	-69.719	J	A	A	A	SMA-1 Acc.	
13	GSC7	GSC SITE 7 - BAIE-St-PAUL, QUE	47.442	-70.507	J	C	B	C	SMA-1 Acc.	Depth A1 uncertain?
14	GSC8	GSC SITE 8 - LA MALBAIE, QUE	47.655	-70.153	J	A	B	A	SMA-1 Acc.	Near region B
15	GSC9	GSC SITE 9 - St-PASCAL, QUE	47.426	-69.805	J	A	B	A	SMA-1 Acc.	
16	GSC10	GSC SITE 10 - RIVIERE-QUELLE	47.476	-69.996	J	I	B	A	SMA-1 Acc.	
17	GSC14	GSC SITE 14 - Ste-LUCIE-DE-BEAUREGARD, Q	47.741	-70.017	J	I	B	A	SMA-1 Acc.	
18	GSC16	GSC SITE 16 - CHICOUTIMI-NORD, QUE	48.490	-71.012	J	C	A	A	SMA-1 Acc.	
19	GSC17	GSC SITE 17 - St-ANDRE-DU-LAC, St-JEAN, Q	48.325	-71.992	J	I	A	A	SMA-1 Acc.	
20	GSC20	GSC SITE 20 - LES EBOULEMENTS, QUE	47.550	-70.327	J	I	A	A	SMA-1 Acc.	
21	HNBK	HORNBEAL, TN	36.333	-89.296	H	I	D	D	AC	CL_2 between regions G & H
22	RIDG	RIDGELY, TN	36.264	-89.481	H	I	D	D	AC	
23	OLAP	OLD APPLETON, MISSOURI	37.572	-89.687	H	I	B	A	AC	
24	HL	HOLMES LAKE, N.B (TEM ST) =C9A	46.945	-66.594	B	I	B	B	SMA-1 Acc.	5+ m till
25	MR	MITCHELL LAKE RD, N.B (TEM ST) =CBA	47.034	-66.612	B	I	A	A	SMA-1 Acc.	
26	LL	LOGGIE LODGE I, N.B (TEM ST) =C7A	46.969	-66.529	B	I	A	B	SMA-1 Acc.	
27	IB	INDIAN BROOK I, N.B (TEM ST)	46.978	-66.581	B	I	A	B	AC	
28	BL	BEAR LAKES, N.B, (TEM ST)	46.928	-66.484	B	I	B	B	AC	5+ m till
29	HCL	HICKEY LAKE, N.B, (TEM ST) =GSC19	47.006	-66.546	B	I	A	A	AC	
30	IB2	INDIAN BROOK II, N.B, (TEM ST)	46.993	-66.596	B	I	A	A	AC	
31	2444A	ARKABUTLA DAM, MS, LEFT CREST = 2512	34.760	-90.120	G	H	D	D	AC	CL_2 between regions D & G
32	2444C	ARKABUTLA DAM, MS, RIGHT ABUTMENT = 2514	34.760	-90.120	G	I	D	D	AC	CL_2 between regions D & G
33	2446	TIPTONVILLE, TN: REELFOOT LAKE	36.370	-89.410	H	A	D	D	AC	
34	2420	NEW MADRID, MO: NORANDA ALUMINUM PLANT	36.510	-89.570	H	A	D	D	AC	
35	2415B	WAPPAPELLO DAM, MO, RIGHT TOE = 2417	36.930	-90.270	I	I	B	A	AC	
36	2415A	WAPPAPELLO DAM, MO, RIGHT CREST	36.930	-90.270	I	H	B	D	AC	
37	2444B	ARKABUTLA DAM, MS, LEFT TOE = 2513	34.760	-90.120	G	I	D	D	AC	
38	2627A	FRANKLIN FALLS DAM, NH, DOWNSTREAM	43.447	-71.660	B	I	A	C	AC	
39	2627B	FRANKLIN FALLS DAM, NH, ABUTMENT	43.447	-71.660	B	I	A	B	AC	
40	2627C	FRANKLIN FALLS DAM, NH, CREST	43.447	-71.660	B	I	A	C	AC	
41	2632A	UNION VILLAGE DAM, VERMONT, CREST	43.794	-72.258	J	H	B	C	AC	
42	2632B	UNION VILLAGE DAM, VERMONT, L. ABUTMENT	43.794	-72.258	J	I	B	B	AC	
43	2632C	UNION VILLAGE DAM, VERMONT, DOWNSTREAM	43.794	-72.258	J	I	B	B	AC	
44	2629A	NORTH HARTLAND DAM, VERMONT, ABUTMENT	43.604	-72.361	J	I	B	A	AC	
45	2629B	NORTH HARTLAND DAM, VERMONT, CREST	43.604	-72.361	J	H	B	C	AC	
46	2604	WHITE RIVER JCT. VA HOSPITAL, VERMONT	43.630	-72.330	J	C	B	A	AC	
47	2617	BALL MOUNTAIN DAM, VERMONT, CREST	43.120	-72.780	J	H	A	C	AC	

Table 2.A-2 (continued)
Recording Station Data For ENA Ground Motion Data Base

Rec	ST NO	DESP	ST LAT	ST LON	SZONE	CL 1	CL 2	CL 3	IS TYPE	COMMENTS
48	2630A	NORTH SPRINGFIELD DAM, VERMONT, CREST	43.338	-72.511	J	H	A	C	AC	
49	2630B	NORTH SPRINGFIELD DAM, VERMONT, DOWNSTREAM	43.336	-72.511	J	I	A	A	AC	
50	6097	NW TERR, CANADA: MACKENZIE, NAHANI, ST1	62.200	-124.300	J	I	B	A	AC	
51	6098	NW TERR, CANADA: MACKENZIE, NAHANI, ST2	62.200	-124.300	J	I	B	A	AC	
52	6099	NW TERR, CANADA: MACKENZIE, NAHANI, ST3	62.200	-124.300	J	I	B	A	AC	
53	PPBF	PERRY PLANT, BLDG FOUNDATION	41.800	-81.142	J	D	D	D	AC	
54	RSCO	RISCO, MISSOURI, NCEER	36.550	-89.774	H	I	D	D	AC	
55	HERB	HEBERTVILLE, QUEBEC	48.407	-71.807	J	I	B	B	AC	
56	CHTN	CHARLESTON, MISSOURI REPEATER ST	36.945	-89.392	H	D	D	D	AC	
57	FTPL	FORT PILLON, TN	36.945	-89.392	H	D	D	D	AC	
58	2532	MONTICELLO DAM, SC, CENTER CREST	34.300	-81.333	C	A	A	D	SMA-1 Acc.	CL_3 Narrow valley?
59	WEO	ECTN :WEO	44.020	-78.370	J	B	D	D	MK2-Z Vel.	
60	WBO	ECTN :WBO	45.000	-75.280	J	B	A	A	MK2-Z Vel.	
61	GGN	ECTN :GGN	45.120	-66.820	B	A	A	A	MK2-Z Vel.	
62	SBQ	ECTN :SBQ	45.380	-71.930	J	B	A	A	MK2-Z Vel.	
63	OTT	ECTN :OTT	45.390	-75.720	J	B	A	A	MK1-Z Vel.	
64	FHO	ECTN :FHO	45.460	-76.220	J	B	A	A	MK1-Z Vel.	
65	MNT	ECTN :MNT	45.500	-73.620	J	B	A	A	MK1-Z Vel.	
66	GAC	ECTN :GAC	45.700	-75.480	J	A	A	A	MK1-Z Vel.	
67	LMN	ECTN :LMN	45.850	-64.810	B	A	A	A	MK2-Z Vel.	
68	CKO	ECTN :CKO	45.990	-77.450	J	A	A	A	MK2-Z Vel.	
69	TRQ	ECTN :TRQ	46.220	-74.560	J	A	A	A	MK2-Z Vel.	
70	GNT	ECTN :GNT	46.360	-72.370	J	B	A	A	MK1-Z Vel.	
71	MIQ	ECTN :MIQ	46.370	-75.970	J	A	A	A		
72	SUO	ECTN :SUO	46.400	-81.010	J	A	A	A		
73	SZO	ECTN :SZO	46.440	-81.500	J	A	A	A		
74	GRQ	ECTN :GRQ	46.610	-75.860	J	A	A	A	MK2-Z Vel.	
75	EEO	ECTN :EEO	46.640	-79.070	J	A	A	A		
76	DPQ	ECTN :DPQ	46.680	-72.780	J	A	A	A	MK3-Z Vel.	
77	SWO	ECTN :SWO	46.730	-81.000	J	B	A	A		
78	KLN	ECTN :KLN	46.840	-66.370	B	B	A	A	MK2-Z Vel.	
79	A11	ECTN :A11	47.240	-70.200	J	B	A	A	MK3-3c Vel.	
80	LPQ	ECTN :LPQ	47.340	-70.010	J	B	A	A	MK2-Z Vel.	
81	A54	ECTN :A54	47.460	-70.410	J	A	A	A		
82	A16	ECTN :A16	47.470	-70.010	J	B	A	A	MK3-3c Vel.	
83	EBN	ECTN :EBN	47.540	-68.240	B	B	A	A	MK2-Z Vel.	
84	A61	ECTN :A61	47.690	-70.090	J	B	A	A	MK3-3c Vel.	
85	A21	ECTN :A21	47.700	-69.690	J	B	A	A	MK3-3c Vel.	
86	A64	ECTN :A64	47.830	-69.890	J	A	A	A	MK3-3c Vel.	
87	VDQ	ECTN :VDQ	48.230	-77.970	J	A	A	A		
88	GSQ	ECTN :GSQ	48.910	-67.110	B	B	A	A	MK2-Z Vel.	
89	HTQ	ECTN :HTQ	49.190	-68.390	J	A	A	A	MK2-Z Vel.	
90	MNQ	ECTN :MNQ	50.530	-68.770	J	A	A	A	MK1-Z Vel.	
91	JAQ	ECTN :JAQ	53.800	-75.720	J	A	A	A		
92	LQO	ECTN :LQO	53.810	-77.430	J	A	A	A		
93	C7A	USGS TEMP ST, MIRAMICHI, Loggie Camp	46.970	-66.530	B	B	B	B	DR100-ACC	but with dynamic range of 1
94	C8A	USGS TEMP ST, MIRAMICHI, Prince Camp	47.008	-66.501	B	A	B	B	DR100-ACC	5 meters of till
95	CBA	USGS TEMP ST, MIRAMICHI, Mitchell Rd	47.034	-66.612	B	A	A	A	DR100-ACC	7 m till over 5 m al/sed

Table 2.A-2 (continued)
Recording Station Data For ENA Ground Motion Data Base

Classification Code CL 1		Classification Code CL 2	Classification Code CL 3
I	Instrument shelter	A	Rock
A	One-story building of lightweight construction	Crystalline: igneous or metamorphic rock, including crystalline volcanic rock. Shear wave velocity >1,000 m/s.	Instrument founded on rock material $V_s > 600$ m/s)
B	Two to four story building of lightweight construction, instrument at ground level	B	Shallow Stiff Soil
C	Two to four story building of lightweight construction, instrument at below ground level	Well indurated sedimentary rock: such as Paleozoic/Mesozoic shales, sandstones, limestones, and conglomerates. Shear wave velocity >1,000 m/s.	Soil profile up to 20 meters overlying rock
D	Structure of five or more stories or heavy construction, instrument at lowest level near ground surface	C	Medium Deep Soil
E	Structure of five or more stories or heavy construction, instrument at lowest level below ground surface	Weakly indurated sedimentary or highly weathered igneous/metamorphic rock: such as Tertiary sandstone, shale, and marl, or serpentinite, volcanic ashes, etc. Shear wave velocity 600 to 1,000 m/s.	Soil depth of 20 to 100 meters
H	Instrument located on embankment dam	D	Deep Soil
		Soft rock (C) overlying hard rock (A or B).	Soil Profile greater than 100 meters

Table 2.A-3
ENA Ground Motion Recordings

Rec	EQID	ST_NO	ZONE	SZONE	HYP	EPD	CLD	CH1	CH2	CVI	LP	HPH1	HPH2	HPVT	DT	PGAH1	PGAH2	PGAVT	SOURCE
1	NH750600	2420	H	H	12.73	9.00		268	178	VRT 25.0	0.07	0.07	0.07	0.00	0.0100	0.04300	0.06400	0.03100	NCEER
2	NH760300	2444B	H	G	99.72	99.00		208	118	VRT 25.0	0.07	0.07	0.07	0.00	0.0100	0.04100	0.02200	0.01000	NCEER
3	NH760300	2444A	H	G	99.72	99.00		208	118	VRT 25.0	0.07	0.07	0.07	0.00	0.0100	0.02100	0.01000	0.00600	NCEER
4	NH760300	2444C	H	G	99.72	99.00		208	118	VRT 25.0	0.07	0.07	0.07	0.00	0.0100	0.01100	0.01100	0.00600	NCEER
5	NH760300	2446	H	H	130.55	130.00		250	160	VRT 25.0	0.07	0.07	0.07	0.00	0.0100	0.01100	0.01700	0.01200	NCEER
6	NH760300	2420	H	H	131.55	131.00		268	178	VRT 25.0	0.07	0.07	0.07	0.00	0.0100	0.01300	0.01100	0.01000	NCEER
7	NH760300	2415B	H	I	150.48	150.00		218	128	VRT 25.0	0.07	0.07	0.07	0.00	0.0100	0.01000	0.01200	0.00500	NCEER
8	NH760300	2415A	H	I	150.48	150.00		218	128	VRT 25.0	0.07	0.07	0.07	0.00	0.0100	0.00600	0.00600	0.00500	NCEER
9	NH760301	2444B	H	G	100.28	99.00		208	118	VRT 25.0	0.07	0.07	0.07	0.00	0.0100	0.01000	0.00500	0.00400	NCEER
10	JK780800	2532	C	C	0.61	0.60		180	090	VRT 50.0	2.00	2.00	2.00	0.00	0.0020	0.26500	0.22900	0.08600	NCEER
11	JK781000	2532	C	C	0.28	0.20		180	090	VRT 50.0	2.00	2.00	2.00	0.00	0.0020	0.26100	0.18200	0.10200	NCEER
12	JK781001	2532	C	C	1.41	1.40		180	090	VRT 50.0	2.00	2.00	2.00	0.00	0.0020	0.14200	0.27400	0.17800	NCEER
13	JK791001	2532	C	C	0.80	0.80		180	090	VRT 50.0	2.00	2.00	2.00	0.00	0.0020	0.35400	0.35600	0.17900	NCEER
14	NB820100	EBN	B	B	137.53	137.44				Z	12.0	0.00	0.00	0.80	0.0167			0.00337	ECTN
15	NB820100	LWN	B	B	187.49	187.43				Z	12.0	0.00	0.00	0.80	0.0167			0.00138	ECTN
16	NB820100	GGN	B	B	209.66	209.60				Z	12.0	0.00	0.00	0.80	0.0167			0.00421	ECTN
17	NB820100	GSQ	B	B	215.65	215.60				Z	12.0	0.00	0.00	0.80	0.0167			0.00050	ECTN
18	NB820100	LPQ	B	B	260.40	260.35				Z	12.0	0.00	0.00	0.80	0.0167			0.00042	ECTN
19	NB820100	MNQ	B	B	423.23	423.20				Z	12.0	0.00	0.00	0.80	0.0167			0.00013	ECTN
20	NB820100	SBQ	B	B	447.83	447.80				Z	12.0	0.00	0.00	0.80	0.0167			0.00033	ECTN
21	NB820101	C7A	B	B	12.17	10.64		H1	H2	Z	50.0	6.00	4.30	6.00	0.0050	0.01000	0.01200	0.00700	PE&A
22	NB820101	C8A	B	B	15.09	13.87		H1	H2	Z	50.0	4.00	3.30	7.00	0.0050	0.00800	0.01000	0.00500	PE&A
23	NB820102	C7A	B	B	11.61	11.31		H1	H2	Z	50.0	5.00	6.00	7.00	0.0050	0.01300	0.01100	0.03000	PE&A
24	NB820102	C8A	B	B	14.76	14.53		H1	H2	Z	50.0	4.00	4.00	6.00	0.0050	0.00300	0.00300	0.01000	PE&A
25	NB820103	C7A	B	B	11.76	11.20		H1	H2	Z	50.0	2.00	2.00	2.00	0.0050	0.00900	0.06900	0.07800	PE&A
26	NB820103	C8A	B	B	14.87	14.42		H1	H2	Z	50.0	2.00	2.00	2.00	0.0050	0.01400	0.01400	0.01700	PE&A
27	NB820104	C7A	B	B	10.02	8.50		H1	H2	Z	50.0	3.00	3.00	5.00	0.0050	0.00300	0.02900	0.02000	PE&A
28	NB820104	C8A	B	B	12.77	11.61		H1	H2	Z	50.0	3.00	3.00	3.00	0.0050	0.00300	0.00300	0.00600	PE&A
29	NB820105	C7A	B	B	11.60	11.29		H1	H2	Z	50.0	5.00	5.00	11.00	0.0050	0.00500	0.00800	0.00700	PE&A
30	NB820106	C8A	B	B	5.97	0.78		H1	H2	Z	50.0	3.00	4.00	6.00	0.0050	0.02200	0.03400	0.01500	PE&A
31	NB820106	C8A	B	B	13.93	12.61		H1	H2	Z	50.0	4.00	4.00	4.00	0.0050	0.00900	0.00600	0.00900	PE&A
32	NB820107	C7A	B	B	10.28	9.06		H1	H2	Z	50.0	6.00	3.00	4.00	0.0050	0.01900	0.02000	0.01300	PE&A
33	FK820100	2627A	B	B	8.35	7.63		225	135	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.14300	0.38500	0.27600	NCEER
34	FK820100	2627B	B	B	8.35	7.63		045	315	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.29300	0.55000	0.17600	NCEER
35	FK820100	2627C	B	B	8.35	7.63		045	315	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.12600	0.31300	0.11700	NCEER
36	FK820100	2632A	B	J	62.45	62.36		245	155	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.02300	0.02600	0.02400	NCEER
37	FK820100	2632B	B	J	62.45	62.36		245	155	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.01000	0.00700	0.00600	NCEER
38	FK820100	2632C	B	J	62.45	62.36		245	155	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.03800	0.02300	0.02900	NCEER
39	FK820100	2629A	B	J	62.69	62.60		015	285	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.03800	0.03900	0.01700	NCEER
40	FK820100	2629B	B	J	62.69	62.60		015	285	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.01100	0.00700	0.00400	NCEER
41	FK820100	2604	B	J	60.85	60.75		270	180	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.01500	0.03200	0.02200	NCEER
42	FK820100	2617	B	J	104.73	104.67		030	300	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.00900	0.01000	0.01200	NCEER
43	FK820100	2630A	B	J	76.05	75.97		275	185	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.02500	0.02200	0.02300	NCEER
44	FK820100	2630B	B	J	76.05	75.97		275	185	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.03200	0.02300	0.01400	NCEER

Table 2.A-3 (continued)
ENA Ground Motion Recordings

Rec	EQID	ST_NO	ZONE	SZONE	HYP	EPD	CID	CH1	CH2	CVT	LP	HPH1	HPH2	HPVT	DT	PGAH1	PGAH2	PGAVI	SOURCE
45	FK820100	SBQ	B	J	209.38	209.26				Z	12.0	0.80	0.80	0.80	0.0167			0.00033	ECTN
46	FK820100	MNT	B	J	272.18	272.09				Z	12.0	0.80	0.80	0.80	0.0167			0.00024	ECTN
47	FK820100	GNT	B	J	322.24	322.17				Z	12.0	0.80	0.80	0.80	0.0167			0.00007	ECTN
48	FK820100	WBO	B	J	335.17	335.09				Z	12.0	0.80	0.80	0.80	0.0167			0.00011	ECTN
49	FK820100	OTT	B	J	386.64	386.58				Z	12.0	0.80	0.80	0.80	0.0167			0.00015	ECTN
50	FK820100	LPQ	B	J	443.81	443.75				Z	12.0	0.80	0.80	0.80	0.0167			0.00015	ECTN
51	NB820300	HL	B	B	7.53	6.38	018	288			50.0	1.00	1.00	0.00	0.0050	0.18100	0.34600	0.15400	PE&A
52	NB820300	MR	B	B	5.66	4.00	028	118			50.0	1.00	1.00	0.00	0.0050	0.15200	0.23500	0.58200	PE&A
53	NB820300	LL	B	B	7.21	6.00	099	189			50.0	1.00	1.00	0.00	0.0050	0.29700	0.57500	0.30700	PE&A
54	NB820300	IB	B	B	5.00	3.00	231	321			50.0	1.00	1.00	0.00	0.0050	0.42500	0.41300	0.14600	PE&A
55	NB820300	HCL	B	B	5.72	4.09	099	189		VRT	50.0	0.00	0.00	0.00	0.0050	0.39700	0.18600	0.15300	NCEER
56	NB820300	IB2	B	B	4.08	0.78	321	231		VRT	50.0	0.00	0.00	0.00	0.0050	0.28900	0.34100	0.08800	NCEER
57	NB820300	KLN	B	B	25.41	24.92				Z	12.0	0.80	0.80	0.80	0.1670			0.00268	ECTN
58	NB820300	EBN	B	B	137.53	137.44				Z	12.0	0.80	0.80	0.80	0.1670			0.00067	ECTN
59	NB820300	GGN	B	B	209.66	209.60				Z	12.0	0.80	0.80	0.80	0.1670			0.00053	ECTN
60	NB820300	GSQ	B	B	215.65	215.59				Z	12.0	0.80	0.80	0.80	0.1670			0.00019	ECTN
61	NB820300	LPQ	B	J	260.40	260.35				Z	12.0	0.80	0.80	0.80	0.1670			0.00006	ECTN
62	NB820300	MNQ	B	J	423.23	423.20				Z	12.0	0.80	0.80	0.80	0.1670			0.00006	ECTN
63	NB820300	GNT	B	J	445.63	445.60				Z	12.0	0.80	0.80	0.80	0.1670			0.00002	ECTN
64	NB820300	SBQ	B	J	447.83	447.80				Z	12.0	0.80	0.80	0.80	0.1670			0.00005	ECTN
65	NB820500	LL	B	B	7.00	7.00	090	189		VRT	50.0	1.00	1.00	0.00	0.0050	0.11700	0.14900	0.00700	NCEER
66	NB820500	HCL	B	B	4.09	4.09	099	189		VRT	35.0	0.20	0.20	0.20	0.0050	0.11000	0.11100	0.06800	NCEER
67	NB820600	KLN	B	B	48.56	47.90				Z	12.0	0.80	0.80	0.80	0.1670			0.00328	ECTN
68	NB820600	EBN	B	B	113.98	113.70				Z	12.0	0.80	0.80	0.80	0.1670			0.00105	ECTN
69	NB820600	LMN	B	B	208.67	208.52				Z	12.0	0.80	0.80	0.80	0.1670			0.00022	ECTN
70	NB820600	GGN	B	B	210.41	210.26				Z	12.0	0.80	0.80	0.80	0.1670			0.00084	ECTN
71	NB820600	GSQ	B	B	211.61	211.45				Z	12.0	0.80	0.80	0.80	0.1670			0.00009	ECTN
72	NB820600	LPQ	B	J	234.16	234.02				Z	12.0	0.80	0.80	0.80	0.1670			0.00011	ECTN
73	NB820600	HTQ	B	J	264.87	264.75				Z	12.0	0.80	0.80	0.80	0.1670			0.00007	ECTN
74	NB820600	MNQ	B	J	413.28	413.20				Z	12.0	0.80	0.80	0.80	0.1670			0.00003	ECTN
75	NB820600	GNT	B	J	419.51	419.43				Z	12.0	0.80	0.80	0.80	0.1670			0.00002	ECTN
76	NB820600	SBQ	B	J	423.80	423.73				Z	12.0	0.80	0.80	0.80	0.1670			0.00004	ECTN
77	GN831000	WBO	J	J	143.40	143.35				Z	12.0	0.80	0.80	0.80	0.1670			0.00236	ECTN
78	GN831000	MNT	J	J	180.40	180.36				Z	12.0	0.80	0.80	0.80	0.1670			0.00151	ECTN
79	GN831000	OTT	J	J	198.70	198.66				Z	12.0	0.80	0.80	0.80	0.1670			0.00130	ECTN
80	GN831000	SBQ	J	J	243.42	243.39				Z	12.0	0.80	0.80	0.80	0.1670			0.00120	ECTN
81	GN831000	TRQ	J	J	254.56	254.53				Z	12.0	0.80	0.80	0.80	0.1670			0.00135	ECTN
82	GN831000	GNT	J	J	306.67	306.65				Z	12.0	0.80	0.80	0.80	0.1670			0.00046	ECTN
83	GN831000	GRQ	J	J	322.34	322.32				Z	12.0	0.80	0.80	0.80	0.1670			0.00054	ECTN
84	GN831000	WEO	J	J	329.62	329.59				Z	12.0	0.80	0.80	0.80	0.1670			0.00165	ECTN
85	GN831000	CKO	J	J	339.44	339.42				Z	12.0	0.80	0.80	0.80	0.1670			0.00052	ECTN
86	GN831000	LPQ	J	J	501.29	501.27				Z	12.0	0.80	0.80	0.80	0.1670			0.00024	ECTN

Table 2.A-3 (continued)
ENA Ground Motion Recordings

Rec	EQID	ST NO	ZONE	SZONE	HYP	EPD	CID	CH1	CH2	CVT	LP	HPH1	HPH2	HPVT	DT	PGAH1	PGAH2	PGAVT	SOURCE
87	NH851100	6098	J	J	18.82	5.50	8.10	240	330	VRT	50.0	0.17	0.50	0.00	0.0050	0.46000	0.38200	0.25400	NETS
88	NH851200	6097	J	J	9.68	7.60	6.00	010	280	VRT	50.0	0.17	0.17	0.17	0.0050	1.10100	1.34500	2.36700	NETS
89	NH851200	6098	J	J	9.53	7.40	8.00	330	240	VRT	50.0	0.17	0.17	0.17	0.0050	0.39000	0.54500		NETS
90	NH851200	6099	J	J	23.38	22.60	16.00	360	270	VRT	50.0	0.17	0.17	0.17	0.0050	0.19400	0.18600	0.18100	NETS
91	NH851201	6099	J	J	18.88	17.90	19.20	270	360	VRT	50.0	0.50	0.50	0.00	0.0050	0.08900	0.10500	0.07400	NETS
92	NO860100	PPBF	J	J	17.63	16.75	180	270	360	VRT	50.0	0.00	0.00	0.00	0.0050	0.17900	0.10000	0.10400	NCEER
93	A871111	GAC	J	J	25.32	14.00	E			Z	12.0	0.80	0.80	0.80	0.0333	0.00053	0.00055	0.00061	ECTN
94	A871111	OTT	J	J	56.36	52.26				Z	12.0	0.80	0.80	0.80	0.0167			0.00018	ECTN
95	A871111	TRQ	J	J	80.42	77.61				Z	12.0	0.80	0.80	0.80	0.0167			0.00016	ECTN
96	A871111	WBO	J	J	88.88	86.34				Z	12.0	0.80	0.80	0.80	0.0167			0.00013	ECTN
97	A871111	GRQ	J	J	103.24	101.06				Z	12.0	0.80	0.80	0.80	0.0167			0.00008	ECTN
98	A871111	MNT	J	J	138.43	136.81				Z	12.0	0.80	0.80	0.80	0.0167			0.00005	ECTN
99	A871111	CKO	J	J	166.59	165.25				Z	12.0	0.80	0.80	0.80	0.0167			0.00004	ECTN
100	A880310	GRQ	J	J	35.43	33.34				Z	12.0	0.80	0.80	0.80	0.0167			0.00084	ECTN
101	A880310	GAC	J	J	73.60	72.61	E			Z	12.0	0.80	0.80	0.80	0.3330	0.00020	0.00031	0.00013	ECTN
102	A880310	TRQ	J	J	87.11	86.28				Z	12.0	0.80	0.80	0.80	0.0167			0.00024	ECTN
103	A880310	OTT	J	J	106.32	105.64				Z	12.0	0.80	0.80	0.80	0.0167			0.00023	ECTN
104	A880310	CKO	J	J	142.91	142.41				Z	12.0	0.80	0.80	0.80	0.0167			0.00015	ECTN
105	A880310	MNT	J	J	184.31	183.92				Z	12.0	0.80	0.80	0.80	0.0167			0.00008	ECTN
106	MS880800	MSNA	J	J	13.79	10.00	245	335	335	VRT	50.0	0.00	0.00	0.00	0.0050	0.00600	0.00900	0.00700	NCEER
107	SG881100	DKY	J	B	198.58	196.60	000	090	090	VRT	50.0	0.00	0.00	0.00	0.0050	0.00100	0.00100	0.00100	NCEER
108	SG881100	A61	J	J	100.33	96.05	E	N	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00172	0.00159	0.00088	ECTN
109	SG881100	A64	J	J	106.98	102.98	E	N	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00069	0.00121	0.00052	ECTN
110	SG881100	A16	J	J	118.78	115.18	E	N	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00086	0.00070	0.00054	ECTN
111	SG881100	A21	J	J	125.58	122.18	E	N	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00113	0.00162	0.00066	ECTN
112	SG881100	A11	J	J	127.34	123.99	E			Z	12.0	0.80	0.80	0.80	0.0125	0.00058	0.00060	0.00057	ECTN
113	SG881100	LPQ	J	J	128.28	124.96				Z	12.0	0.80	0.80	0.80	0.0167			0.00066	ECTN
114	SG881100	DPQ	J	J	202.29	200.20				Z	12.0	0.80	0.80	0.80	0.0167			0.00109	ECTN
115	SG881100	EBN	J	B	232.13	230.31				Z	12.0	0.80	0.80	0.80	0.0167			0.00013	ECTN
116	SG881100	HTQ	J	J	239.26	237.49				Z	12.0	0.80	0.80	0.80	0.0167			0.00020	ECTN
117	SG881100	S8Q	J	J	311.94	310.59				Z	12.0	0.80	0.80	0.80	0.0167			0.00017	ECTN
118	SG881100	GSQ	J	B	314.58	313.24				Z	12.0	0.80	0.80	0.80	0.0167			0.00006	ECTN
119	SG881100	TRQ	J	J	332.11	330.84				Z	12.0	0.80	0.80	0.80	0.0167			0.00032	ECTN
120	SG881100	MNT	J	J	346.58	345.36				Z	12.0	0.80	0.80	0.80	0.0167			0.00019	ECTN
121	SG881100	GRQ	J	J	390.32	389.24				Z	12.0	0.80	0.80	0.80	0.0167			0.00031	ECTN
122	SG881100	KLN	J	B	391.06	389.99				Z	12.0	0.80	0.80	0.80	0.0167			0.00003	ECTN
123	SG881100	OTT	J	J	460.36	459.45				Z	12.0	0.80	0.80	0.80	0.0167			0.00012	ECTN
124	SG881100	WBO	J	J	467.95	467.05				Z	12.0	0.80	0.80	0.80	0.0167			0.00009	ECTN
125	SG881100	GGN	J	B	473.75	472.87				Z	12.0	0.80	0.80	0.80	0.0167			0.00003	ECTN
126	SG881101	EMME	J	B	472.29	471.40	335	065	065	VRT	50.0	0.00	0.00	0.00	0.0050	0.00100		0.00200	NCEER
127	SG881101	MIME	J	B	360.77	359.60	326	056	056	VRT	50.0	0.00	0.00	0.00	0.0050	0.00100	0.00100	0.00100	NCEER
128	SG881101	ISFL	J	B	325.79	324.50	177	267	267	VRT	50.0	0.00	0.00	0.00	0.0050	0.00500	0.00600	0.00400	NCEER
129	SG881101	DKY	J	B	196.95	194.80	000	090	090	VRT	50.0	0.00	0.00	0.00	0.0050	0.06300	0.09100	0.03300	NCEER

Table 2.A-3 (continued)
ENA Ground Motion Recordings

Rec	EQID	ST_NO	ZONE	SZONE	HYP	EPD	CLD	CH1	CH2	CVT	LP	HPH1	HPH2	HPVT	DT	PGAH1	PGAH2	PGAVT	SOURCE
130	SG881101	GSC1	J	J	117.56	113.93		000	270	VRT 50.0	50.0	0.50	0.50	0.50	0.0050	0.12100	0.09700	0.06200	NCEER
131	SG881101	GSC2	J	J	167.01	164.47		051	321	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.05100	0.05100	0.02000	NCEER
132	SG881101	GSC5	J	J	113.08	109.30		097	007	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.02700	0.00200	0.05300	NCEER
133	SG881101	GSC7	J	J	95.61	91.11		175	085	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.12500	0.17400	0.12400	NCEER
134	SG881101	GSC8	J	J	97.50	93.09		063	333	VRT 50.0	0.33	0.33	0.33	0.33	0.0050	0.12400	0.06000	0.06800	NCEER
135	SG881101	GSC9	J	J	132.53	129.32		000	270	VRT 50.0	0.40	0.40	0.40	0.40	0.0050	0.04600	0.05600	0.03700	NCEER
136	SG881101	GSC10	J	J	118.11	114.49		000	270	VRT 50.0	0.40	0.40	0.40	0.40	0.0050	0.04000	0.05700	0.02300	NCEER
137	SG881101	GSC14	J	J	101.34	97.10		000	270	VRT 50.0	0.40	0.40	0.40	0.40	0.0050	0.01400	0.02300	0.02300	NCEER
138	SG881101	GSC16	J	J	51.91	43.05		214	124	VRT 50.0	0.67	0.67	0.67	0.67	0.0050	0.10700	0.13100	0.10200	NCEER
139	SG881101	GSC17	J	J	70.35	64.09		000	270	VRT 50.0	0.80	0.80	0.80	0.80	0.0050	0.15600	0.09100	0.04500	NCEER
140	SG881101	GSC20	J	J	94.97	90.43		000	270	VRT 50.0	0.50	0.50	0.50	0.50	0.0050	0.12500	0.10200	0.02340	NCEER
141	SG881101	SBQ	J	J	311.13	309.78				Z	12.0	0.80	0.80	0.80	0.0167			0.00546	ECTN
142	SG881101	GSQ	J	B	313.51	312.16				Z	12.0	0.80	0.80	0.80	0.0167			0.00326	ECTN
143	SG881101	TRQ	J	J	332.58	331.31				Z	12.0	0.80	0.80	0.80	0.0167			0.00438	ECTN
144	SG881101	KLN	J	B	389.30	388.21				Z	12.0	0.80	0.80	0.80	0.0167			0.00161	ECTN
145	SG881101	GRQ	J	J	391.23	390.15				Z	12.0	0.80	0.80	0.80	0.0167			0.00299	ECTN
146	SG881101	WBO	J	J	468.16	467.26				Z	12.0	0.80	0.80	0.80	0.0167			0.00283	ECTN
147	SG881101	GGN	J	B	471.91	471.02				Z	12.0	0.80	0.80	0.80	0.0167			0.00197	ECTN
148	NM881200	HNBK	H	H	20.81	20.20		090	000	VRT 0.0	0.00	0.00	0.00	0.00	0.0050	0.00200	0.00100	0.00200	NCEER
149	NM881200	RIDG	H	H	11.18	10.00		097	007	VRT 0.0	0.00	0.00	0.00	0.00	0.0050	0.01300	0.01000	0.00900	NCEER
150	NM890400	HNBK	H	H	56.69	55.80		090	000	VRT 0.0	0.00	0.00	0.00	0.00	0.0050	0.00500	0.00500	0.00300	NCEER
151	NM890400	RIDG	H	H	39.87	38.60		097	007	VRT 0.0	0.00	0.00	0.00	0.00	0.0050	0.00900	0.00700	0.01100	NCEER
152	NM890400	OLAP	H	H	174.19	173.90		070	340	VRT 0.0	0.00	0.00	0.00	0.00	0.0050	0.00400	0.00300	0.00300	NCEER
153	MS890700	MSNA	J	J	11.72	7.50		335	245	VRT 0.0	0.00	0.00	0.00	0.00	0.0050	0.00300	0.00600	0.00500	NCEER
154	A900421	A16	J	J	14.05	10.26		E	N	Z	12.0	0.80	0.80	0.80	0.0100	0.00098	0.00191	0.00047	ECTN
155	A900421	A61	J	J	18.06	15.29		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00217	0.00139	0.00067	ECTN
156	A900421	A54	J	J	29.16	27.53		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00024	0.00032	0.00044	ECTN
157	A900421	A21	J	J	34.19	32.81		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00061	0.00047	0.00027	ECTN
158	A900421	A64	J	J	34.94	33.60		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00045	0.00024	0.00038	ECTN
159	A900421	A11	J	J	37.38	36.13		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00020	0.00012	0.00008	ECTN
160	A900421	EBN	J	B	137.60	137.26				Z	12.0	0.80	0.80	0.80	0.0167			0.00003	ECTN
161	CG900900	OLAP	H	H	47.73	46.20		340	070	VRT 0.0	0.00	0.00	0.00	0.00	0.0050	0.02100	0.01900	0.03400	NCEER
162	CG900900	RIDG	H	H	101.02	100.30		007	097	VRT 0.0	0.00	0.00	0.00	0.00	0.0050	0.00500	0.00500	0.00400	NCEER
163	CG900900	RSCO	H	H	71.42	70.40		000	090	VRT 0.0	0.00	0.00	0.00	0.00	0.0050	0.00700	0.01500	0.00600	NCEER
164	ML901000	MSNA	J	J	177.48	177.00		245	335	VRT 0.0	0.00	0.00	0.00	0.00	0.0050	0.00600	0.00300	0.00200	NCEER
165	ML901000	GRQ	J	J	26.70	23.32				Z	12.0	0.80	0.80	0.80	0.0167			0.09164	ECTN
166	ML901000	TRQ	J	J	86.59	85.61				Z	12.0	0.80	0.80	0.80	0.0167			0.02936	ECTN
167	ML901000	OTT	J	J	124.37	123.69				Z	12.0	0.80	0.80	0.80	0.0167			0.00140	ECTN
168	ML901000	CKO	J	J	153.60	153.05				Z	12.0	0.80	0.80	0.80	0.0167			0.01101	ECTN
169	ML901000	WBO	J	J	169.02	168.52				Z	12.0	0.80	0.80	0.80	0.0167			0.01004	ECTN
170	ML901000	MNT	J	J	189.41	188.97				Z	12.0	0.80	0.80	0.80	0.0167			0.00105	ECTN
171	ML901000	DPQ	J	J	216.66	216.27				Z	12.0	0.80	0.80	0.80	0.0167			0.00206	ECTN
172	ML901000	EEO	J	J	265.85	265.53				Z	12.0	0.80	0.80	0.80	0.0167			0.00042	ECTN
173	ML901000	SBQ	J	J	309.98	309.71				Z	12.0	0.80	0.80	0.80	0.0167			0.00044	ECTN
174	ML901000	WEO	J	J	350.81	350.57				Z	12.0	0.80	0.80	0.80	0.0167			0.00093	ECTN

Table 2.A-3 (continued)
ENA Ground Motion Recordings

Rec	EQID	ST_NO	ZONE	SZONE	HYP	EPD	CLD	CH1	CH2	CVT	LP	HPH1	HPH2	HPVT	DT	PGAH1	PGAH2	PGAVI	SOURCE
175	ML901000	A54	J	J	407.87	407.67		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00029	0.00065	0.00040	ECTN
176	ML901000	SWO	J	J	413.17	412.96				Z	12.0	0.80	0.80	0.80	0.0167			0.00015	ECTN
177	ML901000	SUO	J	J	414.55	414.35				Z	12.0	0.80	0.80	0.80	0.0167			0.00011	ECTN
178	ML901000	A11	J	J	418.59	418.39		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00025	0.00018	0.00026	ECTN
179	ML901000	A16	J	J	437.43	437.24		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00024	0.00022	0.00028	ECTN
180	ML901000	A61	J	J	437.49	437.29		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00029	0.00033	0.00015	ECTN
181	ML901000	SZO	J	J	451.78	451.59				Z	12.0	0.80	0.80	0.80	0.0167			0.00011	ECTN
182	ML901000	A64	J	J	456.18	455.99		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00023	0.00024	0.00016	ECTN
183	ML901000	A21	J	J	466.68	466.50		E	N	Z	12.0	0.80	0.80	0.80	0.0125	0.00040	0.00522	0.00024	ECTN
184	NM901100	RIDG	H	H	34.27	33.07		007	097	VRT	0.0	0.00	0.00	0.00	0.0050	0.00600	0.00800	0.00400	NCEER
185	SQ901200	HERB	J	J	109.29	107.80		000	090	VRT	0.0	0.00	0.00	0.00	0.0050	0.00600	0.00700	0.00500	NCEER
186	NM910500	RSCO	H	H	2.30	2.30		000	090	VRT	0.0	0.00	0.00	0.00	0.0050	0.00300	0.00200	0.00200	NCEER
187	NM910501	OLAP	H	H	114.22	114.00		340	070	VRT	0.0	0.00	0.00	0.00	0.0050	0.00800	0.00700	0.00900	NCEER
188	NM910501	RIDG	H	H	39.84	39.20		007	097	VRT	0.0	0.00	0.00	0.00	0.0050	0.00400	0.00500	0.00600	NCEER
189	NM910501	RSCO	H	H	7.60	2.70		000	090	VRT	0.0	0.00	0.00	0.00	0.0050	0.12700	0.09800	0.10800	NCEER
190	NM910501	CHTN	H	H	54.76	54.30		000	090	VRT	0.0	0.00	0.00	0.00	0.0050	0.00200	0.00200	0.00100	NCEER
191	NM910501	FTPL	H	H	99.65	99.40		310	040	VRT	0.0	0.00	0.00	0.00	0.0050	0.00800	0.00700	0.00600	NCEER

Table 2.A-4

Five-Percent Damped Spectral Accelerations (in g) for ENA Ground Motion Records

Rec	EQID	ST_NO	COMP	AMAX	F1	F1.5	F2.5	F3.45	F5	F7.5	F10	F15	F25	F34	F50	F75	F100
1	NM750600	2420	268	0.04363	0.00059	0.00113	0.00432	0.00878	0.01663	0.03434	0.03299	0.08161	0.15045				
2	NM750600	2420	178	0.06473	0.00201	0.00324	0.00864	0.01857	0.03886	0.07145	0.08950	0.11668	0.10934				
3	NM750600	2420	VRT	0.03140	0.00171	0.00229	0.00396	0.00597	0.00944	0.01703	0.02343	0.05091	0.08762				
4	NM760300	2444B	208	0.04190	0.00269	0.00418	0.01026	0.02188	0.08402	0.13867	0.08328	0.05776	0.04113				
5	NM760300	2444B	118	0.02059	0.00278	0.00505	0.00950	0.01418	0.02661	0.06956	0.05723	0.05470	0.04762				
6	NM760300	2444C	VRT	0.00982	0.00102	0.00125	0.00241	0.00364	0.01026	0.04316	0.02692	0.01534	0.01815				
7	NM760300	2444A	208	0.02100	0.00224	0.00407	0.00913	0.01813	0.04539	0.08100	0.04083	0.03880	0.02308		0.02181	0.02143	0.02150
8	NM760300	2444A	118	0.00995	0.00256	0.00383	0.00586	0.01169	0.03045	0.03048	0.02440	0.02103	0.01386		0.00996	0.01011	0.01014
9	NM760300	2444A	VRT	0.00566	0.00174	0.00229	0.00263	0.00387	0.00844	0.01825	0.01069	0.01334	0.01326		0.00662	0.00634	0.00582
10	NM760300	2444C	208	0.01131	0.00206	0.00286	0.00591	0.01270	0.02511	0.03309	0.02596	0.03724	0.01910				
11	NM760300	2444C	118	0.01091	0.00086	0.00145	0.00497	0.01055	0.02931	0.02355	0.01793	0.03126	0.02008				
12	NM760300	2444C	VRT	0.00606	0.00112	0.00171	0.00344	0.00405	0.01337	0.01390	0.01369	0.01896	0.01496				
13	NM760300	2446	250	0.01121	0.00133	0.00134	0.00288	0.00646	0.02179	0.02640	0.03254	0.04059	0.03903				
14	NM760300	2446	160	0.01764	0.00175	0.00269	0.00640	0.01486	0.03888	0.04097	0.03772	0.06006	0.03330				
15	NM760300	2446	VRT	0.01295	0.00205	0.00249	0.00338	0.00409	0.00691	0.01359	0.01529	0.02872	0.02071				
16	NM760300	2420	268	0.01274	0.00285	0.00446	0.01006	0.01613	0.01640	0.03785	0.02811	0.03575	0.01681				
17	NM760300	2420	178	0.01162	0.00074	0.00120	0.00421	0.00863	0.01178	0.01580	0.02452	0.04309	0.01787				
18	NM760300	2420	VRT	0.00985	0.00162	0.00249	0.00409	0.00466	0.00507	0.00634	0.00874	0.02614	0.01902				
19	NM760300	2415B	218	0.01016	0.00042	0.00066	0.00223	0.00658	0.00625	0.01421	0.02227	0.03871	0.02000				
20	NM760300	2415B	128	0.01193	0.00075	0.00081	0.00241	0.00510	0.00715	0.01509	0.01510	0.02612	0.02681				
21	NM760300	2415B	VRT	0.00547	0.00091	0.00128	0.00172	0.00256	0.00277	0.00612	0.00632	0.00761	0.00870		0.00839	0.00701	0.00606
22	NM760300	2415A	218	0.00606	0.00192	0.00243	0.00608	0.00812	0.00826	0.00612	0.00632	0.00761	0.00870		0.00931	0.00775	0.00628
23	NM760300	2415A	128	0.00625	0.00139	0.00217	0.00658	0.01659	0.00857	0.00811	0.01192	0.00957	0.01362		0.00618	0.00537	0.00478
24	NM760300	2415A	VRT	0.00466	0.00118	0.00155	0.00141	0.00239	0.00429	0.00436	0.00504	0.01139	0.00882				
25	NM760301	2444B	208	0.00964	0.00070	0.00126	0.00293	0.00620	0.01581	0.02141	0.02278	0.02076	0.01481				
26	NM760301	2444B	118	0.00537	0.00083	0.00095	0.00183	0.00286	0.00709	0.01231	0.02278	0.02076	0.01481				
27	NM760301	2444B	VRT	0.00360	0.00064	0.00055	0.00115	0.00173	0.00343	0.01117	0.00757	0.00457	0.00430				
28	NM760301	2532	180	0.26499	0.00267	0.00501	0.01185	0.02083	0.05574	0.10271	0.18631	0.44457	0.79857		0.34703	0.26922	0.27193
29	NM760301	2532	090	0.22901	0.00771	0.01553	0.06689	0.12036	0.20483	0.32192	0.42025	0.81636	0.45125		0.24305	0.23506	0.23231
30	NM760301	2532	VRT	0.08642	0.00459	0.00933	0.03459	0.06340	0.10886	0.17325	0.16096	0.22758	0.18693		0.09349	0.08938	0.08804
31	NM760301	2532	180	0.26115	0.00174	0.00312	0.00720	0.01304	0.02760	0.06499	0.12921	0.38587	0.95300		0.51549	0.30144	0.27763
32	NM760301	2532	090	0.18222	0.00297	0.00548	0.00960	0.01967	0.04549	0.07220	0.14794	0.26600	0.54703		0.43624	0.27818	0.19579
33	NM760301	2532	VRT	0.10189	0.00159	0.00341	0.00888	0.01790	0.04450	0.07000	0.11974	0.22084	0.20934		0.17009	0.13264	0.10455
34	NM760301	2532	180	0.14198	0.00547	0.00969	0.03356	0.07369	0.15304	0.30579	0.36295	0.32469	0.43937		0.30359	0.25451	0.15546
35	NM760301	2532	090	0.27407	0.00573	0.01160	0.03163	0.08683	0.23306	0.30698	0.35863	0.42850	0.37334		0.68579	0.47676	0.29855
36	NM760301	2532	VRT	0.17794	0.00735	0.01802	0.05656	0.11910	0.29153	0.24691	0.27448	0.46548	0.33387		0.24193	0.25360	0.18674
37	NM760301	2532	180	0.35357	0.00447	0.00813	0.01808	0.03227	0.10495	0.13837	0.41916	0.95059	1.23784		0.76788	0.56318	0.36812
38	NM760301	2532	090	0.35653	0.00559	0.00924	0.01735	0.03086	0.09314	0.16546	0.46661	0.50173	1.47150		0.84172	0.56374	0.38546
39	NM760301	2532	VRT	0.17891	0.00245	0.00469	0.01221	0.02602	0.08548	0.11575	0.22862	0.50606	0.37888		0.46599	0.42765	0.19528
40	NM820100	EBN	Z	0.00337	0.00171	0.00207	0.00204	0.00235	0.00395	0.00801	0.00671						
41	NM820100	LMN	Z	0.00138	0.00115	0.00156	0.00235	0.00331	0.00245	0.00192	0.00279						
42	NM820100	GGN	Z	0.00421	0.00058	0.00150	0.00446	0.00737	0.00659	0.00978	0.01069						
43	NM820100	GSQ	Z	0.00050	0.00016	0.00036	0.00061	0.00111	0.00128	0.00120	0.00096						
44	NM820100	LPQ	Z	0.00042	0.00039	0.00054	0.00084	0.00095	0.00096	0.00091	0.00108						
45	NM820100	MNQ	Z	0.00013	0.00008	0.00012	0.00023	0.00028	0.00032	0.00035	0.00036						
46	NM820100	SBQ	Z	0.00033	0.00022	0.00034	0.00054	0.00122	0.00106	0.00074	0.00057						

Table 2.A-4 (continued)
Five-Percent Damped Spectral Accelerations (in g) for ENA Ground Motion Records

Rec	EQID	ST_NO	COMP	AMAX	F1	F1.5	F2.5	F3.45	F5	F7.5	F10	F15	F25	F34	F50	F75	F100
47	NB820101	C7A	H1	0.00995					0.00092	0.00202	0.00504	0.01152	0.02466	0.04086	0.02613		
48	NB820101	C7A	H2	0.01192						0.00256	0.00550	0.01645	0.02713	0.04530	0.02180		
49	NB820101	C7A	Z	0.00692						0.00069	0.00183	0.00703	0.00784	0.01926	0.01725		
50	NB820101	C8A	H1	0.00755					0.00139	0.00269	0.00611	0.01200	0.01466	0.02307	0.01345		
51	NB820101	C8A	H2	0.00958				0.00059	0.00191	0.00483	0.00913	0.02113	0.02851	0.02885	0.01854		
52	NB820101	C8A	Z	0.00458						0.00113	0.00304	0.00855	0.01245	0.01358	0.00835		
53	NB820102	C7A	H1	0.01322					0.00102	0.00261	0.00628	0.01535	0.03494	0.03338	0.02711		
54	NB820102	C7A	H2	0.01051						0.00157	0.00420	0.01280	0.02236	0.03716	0.02069		
55	NB820102	C7A	Z	0.03017						0.00172	0.00300	0.00689	0.02891	0.06226	0.09900		
56	NB820102	C8A	H1	0.00284					0.00081	0.00137	0.00264	0.00634	0.00776	0.01284	0.00787		
57	NB820102	C8A	H2	0.00347					0.00059	0.00112	0.00292	0.00726	0.00895	0.01310	0.00580		
58	NB820102	C8A	Z	0.01022						0.00095	0.00192	0.00589	0.02715	0.03045	0.01905		
59	NB820103	C7A	H1	0.09939					0.01567	0.04091	0.09310	0.20000	0.16575	0.14693	0.15689		
60	NB820103	C7A	H2	0.06935					0.00915	0.02656	0.05874	0.16029	0.13883	0.15318	0.08422		
61	NB820103	C7A	Z	0.07789					0.00349	0.00497	0.00929	0.01888	0.04875	0.08368	0.17205		
62	NB820103	C8A	H1	0.01386					0.01374	0.01206	0.02019	0.03993	0.02981	0.03434	0.02913		
63	NB820103	C8A	H2	0.01425					0.00460	0.00932	0.00863	0.01940	0.03644	0.04146	0.03296		
64	NB820103	C8A	Z	0.01717	0.00016	0.00037	0.00097	0.00072	0.00204	0.00546	0.01006	0.02769	0.04057	0.05945	0.05790		
65	NB820104	C7A	H1	0.02412					0.0072	0.00204	0.01526	0.01976	0.04628	0.07364	0.05755		
66	NB820104	C7A	H2	0.02856			0.00058	0.00118	0.00293	0.00759	0.01748	0.04657	0.05271	0.11638	0.05825		
67	NB820104	C7A	Z	0.02011					0.00125	0.00276	0.00435	0.01601	0.02458	0.04613	0.07276		
68	NB820104	C8A	H1	0.00333					0.00032	0.00078	0.00373	0.00478	0.00692	0.00875	0.00598		
69	NB820104	C8A	H2	0.00312				0.00010	0.00021	0.00053	0.00131	0.00537	0.00412	0.00722	0.00817		
70	NB820104	C8A	Z	0.00571					0.00069	0.00257	0.00470	0.01028	0.01337	0.00965	0.00900		
71	NB820105	C7A	H1	0.00524					0.00060	0.00141	0.00367	0.00951	0.01257	0.01739	0.01143		
72	NB820105	C7A	H2	0.00814					0.00030	0.00087	0.00243	0.00451	0.01679	0.02812	0.02067		
73	NB820105	C7A	Z	0.00695						0.00171	0.00695	0.01581	0.00695	0.01581	0.01952		
74	NB820106	C8A	H1	0.02200				0.00032	0.00064	0.00143	0.00311	0.00972	0.02901	0.05458	0.09534		
75	NB820106	C8A	H2	0.03406					0.00245	0.00592	0.01251	0.03441	0.06748	0.08878	0.09634		
76	NB820106	C8A	Z	0.01457						0.00113	0.00226	0.00576	0.01070	0.03657	0.06096		
77	NB820106	C8A	H1	0.00896					0.00210	0.00612	0.00947	0.01060	0.01666	0.01174	0.01805		
78	NB820106	C8A	H2	0.00571			0.00037	0.00072	0.00267	0.00448	0.00740	0.01139	0.01029	0.01158	0.01112		
79	NB820106	C8A	Z	0.00941					0.00060	0.00173	0.00371	0.00934	0.02388	0.01707	0.01860		
80	NB820107	C7A	H1	0.01922						0.00336	0.00780	0.02143	0.02423	0.03788	0.03444		
81	NB820107	C7A	H2	0.02041				0.00072	0.00154	0.00429	0.01085	0.03320	0.04337	0.05011	0.03898		
82	NB820107	C7A	Z	0.01303					0.00052	0.00115	0.00206	0.00704	0.01701	0.01678	0.05249		
83	NB820107	C8A	H1	0.01036						0.00217	0.01288	0.02145	0.02482	0.03011	0.02482		
84	NB820107	C8A	H2	0.01387					0.00233	0.00569	0.01004	0.02532	0.02589	0.04407	0.02725		
85	NB820107	C8A	Z	0.01182						0.00224	0.00680	0.02303	0.02610	0.03367	0.03367		
86	FK820100	2627A	225	0.14343	0.00664	0.01937	0.10210	0.09203	0.07039	0.10692	0.16643	0.23063	0.32610	0.29153	0.16648		
87	FK820100	2627A	135	0.38518	0.01067	0.01506	0.08797	0.10978	0.18318	0.25321	0.38414	0.85611	0.68824	0.36459	0.58061		
88	FK820100	2627A	VRT	0.27625	0.00433	0.00601	0.02783	0.04142	0.04947	0.09900	0.18769	0.39118	0.54190	0.34237	0.42521		
89	FK820100	2627B	045	0.29327	0.00908	0.01347	0.02746	0.03988	0.07476	0.13573	0.34811	1.20070	0.63744	0.51379	0.34339		
90	FK820100	2627B	315	0.55042	0.01040	0.01499	0.04650	0.07880	0.10662	0.25076	0.81978	1.95471	1.47853	1.11715	0.65736		
91	FK820100	2627B	VRT	0.17624	0.00973	0.02533	0.04769	0.07506	0.06065	0.10742	0.24485	0.36642	0.73655	0.35451	0.21762		

Table 2.A-4 (continued)

Five-Percent Damped Spectral Accelerations (in g) for ENA Ground Motion Records

REC	EQID	ST. NO	COMP	AMAX	F1	F1.5	F2.5	F3.45	F5	F7.5	F10	F15	F25	F34	F50	F75	F100
92	FK820100	2627C	045	0.12636	0.00941	0.01666	0.09940	0.07727	0.11957	0.10892	0.16143	0.30281	0.39404	0.24413	0.27837	0.16594	0.12633
93	FK820100	2627C	315	0.31277	0.01422	0.03133	0.13219	0.16625	0.14536	0.23504	0.40290	0.92469	0.45811	0.41761	0.49552	0.40448	0.33971
94	FK820100	2627C	VRT	0.11652	0.00924	0.01167	0.06518	0.19191	0.12600	0.13115	0.20749	0.33036	0.30920	0.26951	0.14449	0.20589	0.13136
95	FK820100	2632A	245	0.02290	0.00127	0.00202	0.01105	0.02573	0.02622	0.04683	0.06425	0.09787	0.06069	0.03623	0.03020	0.03084	0.02478
96	FK820100	2632A	155	0.02556	0.00155	0.00255	0.01267	0.02804	0.02988	0.05330	0.08131	0.12273	0.05429	0.03824	0.04008	0.03539	0.02754
97	FK820100	2632A	VRT	0.02362	0.00120	0.00143	0.00454	0.01547	0.04090	0.06026	0.06412	0.07502	0.06297	0.04692	0.03708	0.02684	0.02494
98	FK820100	2632B	245	0.00967	0.00088	0.00123	0.00171	0.00344	0.00771	0.01263	0.02473	0.01971	0.01861	0.01181	0.01201		
99	FK820100	2632B	155	0.00686	0.00112	0.00181	0.00240	0.00327	0.00478	0.01035	0.03144	0.01452	0.01476	0.01300	0.00907		
100	FK820100	2632B	VRT	0.00633	0.00123	0.00127	0.00235	0.00425	0.00627	0.01056	0.02129	0.01876	0.00953	0.00823	0.01041		
101	FK820100	2632C	245	0.03773	0.00174	0.00255	0.00545	0.01034	0.03993	0.12126	0.13822	0.07634	0.08522	0.05184	0.04986		
102	FK820100	2632C	155	0.02302	0.00154	0.00223	0.00437	0.00595	0.02096	0.11530	0.06915	0.07770	0.04320	0.03140	0.03256		
103	FK820100	2632C	VRT	0.02946	0.00271	0.00298	0.00405	0.00786	0.01116	0.02577	0.06065	0.13317	0.05741	0.04204	0.04740		
104	FK820100	2629A	015	0.00721	0.00117	0.00132	0.00160	0.00258	0.00571	0.00770	0.01004	0.02612	0.02240	0.01805	0.01409		
105	FK820100	2629A	285	0.00697	0.00087	0.00156	0.00262	0.00228	0.00412	0.00734	0.01151	0.01793	0.02284	0.01865	0.01363		
106	FK820100	2629A	VRT	0.00382	0.00139	0.00165	0.00230	0.00188	0.00383	0.00564	0.00745	0.00899	0.01239	0.00899	0.00718		
107	FK820100	2629B	015	0.03806	0.00284	0.00461	0.01309	0.02919	0.04091	0.11450	0.08621	0.10647	0.10224	0.06279	0.05494	0.05629	0.06138
108	FK820100	2629B	285	0.03893	0.00375	0.00509	0.02814	0.04052	0.03671	0.06659	0.06000	0.07877	0.07050	0.08197	0.06868	0.06012	0.05074
109	FK820100	2629B	VRT	0.01705	0.00279	0.00430	0.00803	0.01314	0.02958	0.03867	0.04563	0.04931	0.02556	0.02660	0.02108	0.02399	0.02215
110	FK820100	2604	270	0.01549	0.00124	0.00173	0.00374	0.00690	0.04392	0.03448	0.05067	0.03928	0.03839	0.03196	0.02264		
111	FK820100	2604	180	0.03160	0.00286	0.00430	0.00871	0.01381	0.03644	0.12338	0.07749	0.07245	0.04972	0.04232	0.03484		
112	FK820100	2604	VRT	0.02223	0.00203	0.00281	0.00408	0.00676	0.01258	0.06556	0.10225	0.06290	0.03640	0.02952	0.02407		
113	FK820100	2617	030	0.00897	0.00244	0.00344	0.01464	0.01560	0.02318	0.03494	0.01705	0.02113	0.01292	0.01406	0.01358	0.01192	0.01022
114	FK820100	2617	300	0.01022	0.00102	0.00259	0.01805	0.01245	0.02316	0.02912	0.03192	0.02637	0.01715	0.01336	0.01620	0.01281	0.01080
115	FK820100	2617	VRT	0.01220	0.00130	0.00144	0.00445	0.01417	0.02525	0.02533	0.01957	0.03767	0.02115	0.01467	0.01290	0.01453	0.01296
116	FK820100	2630A	275	0.02485	0.00163	0.00289	0.01123	0.01723	0.02318	0.06690	0.06351	0.09227	0.05713	0.03962	0.03327	0.02928	0.03259
117	FK820100	2630A	185	0.02196	0.00128	0.00234	0.01566	0.02570	0.02424	0.03604	0.04710	0.06913	0.05312	0.03437	0.02408	0.02405	0.02344
118	FK820100	2630A	VRT	0.02288	0.00109	0.00164	0.00469	0.01459	0.03983	0.02770	0.03099	0.07858	0.03229	0.02958	0.02741	0.02488	0.02508
119	FK820100	2630B	275	0.02425	0.00147	0.00202	0.00262	0.00530	0.01701	0.04145	0.05524	0.07142	0.08131	0.04137	0.03573		
120	FK820100	2630B	185	0.02303	0.00129	0.00182	0.00211	0.00529	0.01253	0.02841	0.03414	0.05209	0.10585	0.04223	0.03814		
121	FK820100	2630B	VRT	0.01392	0.00111	0.00166	0.00179	0.00249	0.00476	0.01035	0.03526	0.04188	0.03181	0.02347	0.01880		
122	FK820100	S8Q	Z	0.00033	0.00002	0.00007	0.00030	0.00042	0.00076	0.00087	0.00075						
123	FK820100	MNT	Z	0.00024	0.00001	0.00004	0.00011	0.00025	0.00051	0.00052	0.00079						
124	FK820100	GNT	Z	0.00007	0.00002	0.00005	0.00009	0.00015	0.00018	0.00017	0.00016						
125	FK820100	W8Q	Z	0.00011	0.00002	0.00002	0.00011	0.00011	0.00018	0.00028	0.00040						
126	FK820100	OTT	Z	0.00015	0.00001	0.00003	0.00013	0.00026	0.00038	0.00040	0.00042						
127	FK820100	LPQ	Z	0.00015	0.00001	0.00003	0.00013	0.00026	0.00038	0.00040	0.00042						
128	NB820300	HL	018	0.18077	0.00139	0.00225	0.00613	0.01378	0.02715	0.08368	0.18372	0.32995	0.31550	0.32923	0.31475		
129	NB820300	HL	288	0.34680	0.00698	0.01155	0.02052	0.02932	0.03909	0.14211	0.20593	0.30832	0.54141	0.85516	1.00950		
130	NB820300	MR	028	0.23594	0.00279	0.00730	0.02541	0.04781	0.13662	0.17013	0.35334	0.37680	0.67029	0.50882	0.62996		
131	NB820300	MR	118	0.15165	0.00427	0.00835	0.02830	0.05531	0.13640	0.11738	0.16639	0.25082	0.52505	0.48936	0.57002		
132	NB820300	LL	099	0.57494	0.01606	0.03006	0.05522	0.09109	0.17116	0.16512	0.26646	0.43896	0.90456	1.93573	0.96104		
133	NB820300	LL	189	0.29763	0.00400	0.00783	0.00955	0.01626	0.03915	0.11141	0.16336	0.35493	0.70879	0.69686	0.94884		
134	NB820300	IB	231	0.41289	0.00935	0.02291	0.04057	0.06340	0.09270	0.09304	0.11302	0.33108	2.02278	0.98203	0.73453		
135	NB820300	IB	321	0.42482	0.00494	0.00938	0.02813	0.04793	0.08973	0.16819	0.22446	0.34237	1.69260	0.76981	0.99224		
136	NB820300	HCL	099	0.39722	0.01385	0.02520	0.04752	0.08643	0.17395	0.14939	0.27145	0.44026	0.81909	1.54723	0.54435		
137	NB820300	HCL	189	0.18639	0.00242	0.00528	0.01313	0.01648	0.03322	0.09605	0.17252	0.36194	0.60909	0.59396	0.27909		
138	NB820300	HCL	VRT	0.15332	0.00677	0.01251	0.02448	0.03375	0.05331	0.08565	0.14732	0.15610	0.25371	0.47391	0.24180		

Table 2.A-4 (continued)

Five-Percent Damped Spectral Accelerations (in g) for ENA Ground Motion Records

Rec	EQID	ST NO	COMP	AMAX	F1	F1.5	F2.5	F3.45	F5	F7.5	F10	F15	F25	F34	F50	F75	F100
139	NB820300	IB2	321	0.28977	0.00413	0.00804	0.02460	0.04329	0.08787	0.16188	0.21227	0.33000	1.55476	0.61967	0.39197		
140	NB820300	IB2	231	0.34150	0.01471	0.02743	0.04384	0.06308	0.09172	0.19056	0.10301	0.29925	1.84102	0.84134	0.42933		
141	NB820300	IB2	VRT	0.08864	0.00227	0.00438	0.00838	0.02055	0.02824	0.03346	0.03901	0.08705	0.34083	0.24057	0.14312		
142	NB820300	KLW	Z	0.00268	0.00043	0.00069	0.00130	0.00231	0.00334	0.00617	0.00710						
143	NB820300	EBW	Z	0.00067	0.00008	0.00017	0.00025	0.00085	0.00092	0.00137	0.00115						
144	NB820300	GGN	Z	0.00053	0.00009	0.00014	0.00049	0.00077	0.00109	0.00123	0.00098						
145	NB820300	GSQ	Z	0.00019	0.00006	0.00028	0.00023	0.00051	0.00040	0.00039	0.00047						
146	NB820300	LPQ	Z	0.00006	0.00002	0.00002	0.00005	0.00007	0.00010	0.00022	0.00013						
147	NB820300	MNQ	Z	0.00006	0.00002	0.00005	0.00009	0.00009	0.00014	0.00017	0.00010						
148	NB820300	GNT	Z	0.00002	0.00003	0.00005	0.00005	0.00005	0.00004	0.00005	0.00004						
149	NB820300	SBQ	Z	0.00005	0.00001	0.00004	0.00005	0.00007	0.00006	0.00013	0.00013						
150	NB820500	LL	090	0.11348	0.00356	0.00665	0.01623	0.02767	0.04944	0.14546	0.32231	0.32252	0.18966	0.16520	0.15057		
151	NB820500	LL	189	0.11677	0.00840	0.01465	0.02652	0.04188	0.06379	0.11240	0.15422	0.13634	0.21255	0.14195	0.17065		
152	NB820500	HCL	099	0.11027	0.00433	0.00738	0.01797	0.03039	0.05236	0.14921	0.32441	0.31638	0.19591	0.18987			
153	NB820500	HCL	189	0.08705	0.00678	0.01169	0.02047	0.03318	0.05259	0.09862	0.14409	0.13111	0.21336	0.18330			
154	NB820500	HCL	VRT	0.06838	0.00072	0.00138	0.00311	0.00489	0.01440	0.03467	0.06711	0.16380	0.20906	0.12837			
155	NB820600	KLW	Z	0.00328	0.00006	0.00017	0.00029	0.00072	0.00122	0.00478	0.00635						
156	NB820600	EBW	Z	0.00105	0.00018	0.00056	0.00107	0.00137	0.00124	0.00113	0.00208						
157	NB820600	LMN	Z	0.00022	0.00004	0.00012	0.00013	0.00015	0.00036	0.00079	0.00045						
158	NB820600	GGN	Z	0.00084	0.00003	0.00007	0.00030	0.00071	0.00118	0.00262	0.00339						
159	NB820600	GSQ	Z	0.00009	0.00003	0.00005	0.00015	0.00028	0.00024	0.00017	0.00018						
160	NB820600	LPQ	Z	0.00011	0.00002	0.00005	0.00012	0.00021	0.00020	0.00019	0.00022						
161	NB820600	HTQ	Z	0.00007	0.00001	0.00004	0.00011	0.00016	0.00022	0.00019	0.00017						
162	NB820600	MNQ	Z	0.00003	0.00001	0.00002	0.00006	0.00007	0.00007	0.00007	0.00008						
163	NB820600	GNT	Z	0.00002	0.00003	0.00003	0.00004	0.00005	0.00006	0.00007	0.00004						
164	NB820600	SBQ	Z	0.00004	0.00001	0.00003	0.00008	0.00008	0.00010	0.00014	0.00012						
165	GN831000	WBO	Z	0.00236	0.00017	0.00047	0.00152	0.00297	0.00350	0.00383	0.00462						
166	GN831000	MNT	Z	0.00151	0.00040	0.00072	0.00142	0.00266	0.00277	0.00473	0.00607						
167	GN831000	OTT	Z	0.00130	0.00024	0.00062	0.00166	0.00281	0.00256	0.00268	0.00545						
168	GN831000	SBQ	Z	0.00120	0.00045	0.00122	0.00175	0.00278	0.00328	0.00331	0.00267						
169	GN831000	TRQ	Z	0.00135	0.00023	0.00065	0.00214	0.00251	0.00342	0.00493	0.00393						
170	GN831000	GNT	Z	0.00046	0.00037	0.00088	0.00105	0.00092	0.00125	0.00112	0.00082						
171	GN831000	GRQ	Z	0.00054	0.00014	0.00028	0.00065	0.00151	0.00234	0.00235	0.00139						
172	GN831000	WEO	Z	0.00165	0.00017	0.00051	0.00097	0.00152	0.00201	0.00532	0.00580						
173	GN831000	CKO	Z	0.00052	0.00015	0.00038	0.00165	0.00178	0.00147	0.00146	0.00148						
174	GN831000	LPQ	Z	0.00024	0.00014	0.00025	0.00061	0.00072	0.00088	0.00051	0.00047						
175	NH851100	6098	240	0.45969	0.01437	0.03069	0.07207	0.18375	0.39524	0.46707	0.48599	0.83636	0.50537	0.48220	0.57348		
176	NH851100	6098	330	0.38152	0.01200	0.02369	0.06345	0.12458	0.28303	0.30887	0.43404	0.68732	0.47502	0.45542	0.40261		
177	NH851200	6097	010	1.10133	0.43843	0.60858	0.85652	1.69336	2.85529	2.18039	2.49706	2.53314	2.08663	1.43900	1.20101		
178	NH851200	6097	280	1.34463	0.48920	0.62527	0.91542	1.34457	2.4915	2.01772	2.79211	2.99394	1.39409	1.69227	1.35955		
179	NH851200	6097	VRT	2.36736	0.40354	0.94573	0.63091	0.63600	2.42642	1.96109	5.41788	4.19072	4.06184	3.00575	2.60391		
180	NH851200	6098	330	0.38978	0.28621	0.57940	0.43325	0.70532	0.53902	0.66701	0.56768	0.69077	0.68850	0.65238	0.49200		
181	NH851200	6098	240	0.54479	0.13446	0.45169	0.46777	0.70532	0.41978	0.36005	0.65824	0.98151	1.16859	1.06409	0.82945		
182	NH851200	6099	360	0.19388	0.02325	0.03485	0.05916	0.11406	0.16390	0.37286	0.32180	0.69502	0.58918	0.36387	0.36319		
183	NH851200	6099	270	0.18594	0.03533	0.06346	0.08840	0.17367	0.18145	0.34308	0.29055	0.72447	0.47826	0.31431	0.49514		
184	NH851200	6099	VRT	0.18149	0.03872	0.03385	0.06846	0.14284	0.22495	0.24516	0.26202	0.36607	0.57841	0.40108	0.33109		
185	NH851201	6099	270	0.08910	0.00380	0.00683	0.01646	0.02992	0.05914	0.13972	0.23605	0.21281	0.19906	0.15501	0.17077		
186	NH851201	6099	360	0.10544	0.00800	0.01118	0.01390	0.03327	0.05724	0.08464	0.16866	0.24481	0.27319	0.20551	0.20251		

Table 2.A-4 (continued)

Five-Percent Damped Spectral Accelerations (in g) for ENA Ground Motion Records

Rec	EQID	ST_NO	COMP	AMAX	F1	F1.5	F2.5	F3.45	F5	F7.5	F10	F15	F25	F34	F50	F75	F100
187	NO860100	PPBF	180	0.17981	0.00611	0.01370	0.03416	0.07939	0.12103	0.11149	0.12843	0.24043	0.62641	0.27549	0.20883		
188	NO860100	PPBF	270	0.10027	0.01005	0.02491	0.04733	0.07565	0.11615	0.11503	0.21425	0.33576	0.24032	0.17350	0.11230		
189	NO860100	PPBF	VRT	0.10393	0.00494	0.01148	0.02270	0.09031	0.22437	0.09440	0.08799	0.13216	0.33050	0.18180	0.11492		
190	A871111	GAC	E	0.00053	0.00002	0.00004	0.00006	0.00011	0.00037	0.00100	0.00110						
191	A871111	GAC	N	0.00055	0.00001	0.00002	0.00006	0.00015	0.00038	0.00093	0.00076						
192	A871111	GAC	Z	0.00061	0.00001	0.00002	0.00005	0.00015	0.00034	0.00093	0.00102						
193	A871111	OTT	Z	0.00018	0.00000	0.00001	0.00008	0.00019	0.00032	0.00039	0.00067						
194	A871111	TRQ	Z	0.00016	0.00000	0.00001	0.00008	0.00017	0.00017	0.00043	0.00058						
195	A871111	WBO	Z	0.00013	0.00000	0.00001	0.00002	0.00005	0.00010	0.00026	0.00045						
196	A871111	GRQ	Z	0.00008	0.00000	0.00001	0.00003	0.00007	0.00013	0.00014	0.00027						
197	A871111	MNT	Z	0.00005	0.00000	0.00001	0.00001	0.00004	0.00006	0.00013	0.00019						
198	A871111	CKO	Z	0.00004	0.00000	0.00001	0.00003	0.00003	0.00005	0.00013	0.00015						
199	A880310	GRQ	Z	0.00084	0.00002	0.00006	0.00016	0.00026	0.00092	0.00224	0.00300						
200	A880310	GAC	E	0.00020	0.00000	0.00001	0.00003	0.00005	0.00017	0.00021	0.00041						
201	A880310	GAC	N	0.00031	0.00000	0.00001	0.00003	0.00006	0.00020	0.00035	0.00052						
202	A880310	GAC	Z	0.00013	0.00000	0.00001	0.00002	0.00006	0.00015	0.00029	0.00027						
203	A880310	TRQ	Z	0.00024	0.00001	0.00001	0.00004	0.00006	0.00018	0.00065	0.00109						
204	A880310	OTT	Z	0.00023	0.00001	0.00001	0.00006	0.00016	0.00027	0.00070	0.00079						
205	A880310	CKO	Z	0.00015	0.00000	0.00003	0.00013	0.00025	0.00026	0.00043	0.00050						
206	A880310	MNT	Z	0.00008	0.00000	0.00001	0.00002	0.00005	0.00011	0.00020	0.00036						
207	MS880800	MSNA	245	0.00663	0.00031	0.00022	0.00041	0.00105	0.00436	0.00986	0.01754	0.01822	0.01887	0.01863	0.01372		
208	MS880800	MSNA	335	0.00935	0.00018	0.00042	0.00098	0.00244	0.00596	0.00868	0.01022	0.01386	0.02527	0.02201	0.01504		
209	MS880800	MSNA	VRT	0.00719	0.00011	0.00018	0.00024	0.00044	0.00115	0.00269	0.00706	0.01339	0.02601	0.01852	0.01022		
210	SG881100	CKY	000	0.00109	0.00018	0.00030	0.00073	0.00103	0.00182	0.00362	0.00256	0.00337	0.00170	0.00145	0.00118		
211	SG881100	CKY	090	0.00116	0.00019	0.00043	0.00091	0.00148	0.00266	0.00295	0.00391	0.00198	0.00195	0.00136	0.00128		
212	SG881100	CKY	VRT	0.00071	0.00021	0.00049	0.00081	0.00066	0.00108	0.00186	0.00193	0.00157	0.00162	0.00128	0.00102		
213	SG881100	A61	E	0.00172	0.00010	0.00019	0.00045	0.00070	0.00177	0.00352	0.00441						
214	SG881100	A61	N	0.00159	0.00010	0.00015	0.00031	0.00068	0.00193	0.00248	0.00401						
215	SG881100	A61	Z	0.00088	0.00011	0.00015	0.00030	0.00060	0.00117	0.00182	0.00271						
216	SG881100	A64	E	0.00069	0.00009	0.00014	0.00018	0.00042	0.00121	0.00166	0.00282						
217	SG881100	A64	N	0.00121	0.00006	0.00016	0.00021	0.00074	0.00119	0.00433	0.00353						
218	SG881100	A64	Z	0.00052	0.00005	0.00013	0.00028	0.00041	0.00086	0.00233	0.00231						
219	SG881100	A16	E	0.00086	0.00005	0.00012	0.00027	0.00063	0.00092	0.00172	0.00296						
220	SG881100	A16	N	0.00070	0.00007	0.00016	0.00037	0.00048	0.00089	0.00196	0.00222						
221	SG881100	A16	Z	0.00054	0.00006	0.00012	0.00013	0.00023	0.00056	0.00116	0.00195						
222	SG881100	A21	E	0.00113	0.00010	0.00017	0.00043	0.00071	0.00160	0.00380	0.00428						
223	SG881100	A21	N	0.00162	0.00006	0.00017	0.00044	0.00071	0.00146	0.00435	0.00499						
224	SG881100	A21	Z	0.00066	0.00009	0.00014	0.00018	0.00036	0.00063	0.00134	0.00223						
225	SG881100	A11	E	0.00058	0.00005	0.00024	0.00062	0.00063	0.00091	0.00213	0.00153						
226	SG881100	A11	N	0.00060	0.00018	0.00045	0.00076	0.00082	0.00078	0.00167	0.00219						
227	SG881100	A11	Z	0.00057	0.00008	0.00014	0.00035	0.00025	0.00067	0.00100	0.00147						
228	SG881100	LPQ	Z	0.00066	0.00011	0.00023	0.00032	0.00079	0.00130	0.00274	0.00238						
229	SG881100	DPQ	Z	0.00109	0.00006	0.00024	0.00055	0.00102	0.00123	0.00169	0.00253						
230	SG881100	EBN	Z	0.00013	0.00005	0.00008	0.00011	0.00023	0.00025	0.00035	0.00039						
231	SG881100	HTQ	Z	0.00020	0.00002	0.00004	0.00008	0.00018	0.00032	0.00082	0.00052						
232	SG881100	SBQ	Z	0.00017	0.00011	0.00015	0.00024	0.00045	0.00051	0.00053	0.00063						
233	SG881100	GSQ	Z	0.00006	0.00005	0.00004	0.00012	0.00010	0.00011	0.00016	0.00014						

Table 2.A-4 (continued)

Five-Percent Damped Spectral Accelerations (in g) for ENA Ground Motion Records

Rec	EQID	ST	NO	COMP	AMAX	F1	F1.5	F2.5	F3.45	F5	F7.5	F10	F15	F25	F34	F50	F75	F100
234	SG881100	TRQ		Z	0.00032	0.00004	0.00014	0.00043	0.00058	0.00074	0.00104	0.00074						
235	SG881100	MNT		Z	0.00019	0.00003	0.00008	0.00015	0.00049	0.00047	0.00052	0.00049						
236	SG881100	GRQ		Z	0.00031	0.00004	0.00015	0.00036	0.00065	0.00118	0.00086	0.00049						
237	SG881100	KLW		Z	0.00003	0.00012	0.00010	0.00004	0.00003	0.00004	0.00004	0.00003						
238	SG881100	OTT		Z	0.00012	0.00027	0.00018	0.00013	0.00012	0.00013	0.00013	0.00012						
239	SG881100	WBO		Z	0.00009	0.00021	0.00013	0.00009	0.00009	0.00009	0.00009	0.00009						
240	SG881100	GGN		Z	0.00003	0.00006	0.00006	0.00003	0.00003	0.00003	0.00003	0.00003						
241	SG881101	PAL		N00	0.00059	0.00105	0.00162	0.00185	0.00127	0.00088	0.00074	0.00066	0.00076	0.00075	0.00077	0.00077	0.00077	0.00077
242	SG881101	PAL		N90	0.00070	0.00102	0.00170	0.00253	0.00204	0.00136	0.00101	0.00086	0.00090	0.00083	0.00091	0.00087	0.00087	0.00087
243	SG881101	PAL		VRT	0.00076	0.00139	0.00192	0.00149	0.00120	0.00092	0.00082	0.00080	0.00087	0.00091	0.00085	0.00089	0.00089	0.00089
244	SG881101	NEWC		N15	0.00221	0.00118	0.00305	0.00552	0.00540	0.00496	0.00391	0.00371	0.00340	0.00460	0.00254	0.00237	0.00237	0.00237
245	SG881101	NEWC		N10	0.00276	0.00153	0.00506	0.00720	0.00617	0.00423	0.00468	0.00431	0.00463	0.00468	0.00351	0.00291	0.00291	0.00291
246	SG881101	NEWC		VRT	0.00173	0.00077	0.00240	0.00510	0.00506	0.00371	0.00298	0.00272	0.00219	0.00253	0.00216	0.00213	0.00213	0.00213
247	SG881101	JORD		O10	0.00333	0.00087	0.00175	0.00582	0.00603	0.00843	0.00847	0.01149	0.00508	0.00453	0.00378	0.00342	0.00342	0.00342
248	SG881101	JORD		100	0.00370	0.00164	0.00292	0.00918	0.00878	0.01061	0.01253	0.01398	0.00729	0.00458	0.00430	0.00410	0.00410	0.00410
249	SG881101	JORD		VRT	0.00173	0.00074	0.00186	0.00308	0.00413	0.00446	0.00402	0.00488	0.00526	0.00421	0.00203	0.00207	0.00207	0.00207
250	SG881101	MSNA		N24	0.00560	0.00186	0.00361	0.00719	0.00848	0.01519	0.02025	0.01094	0.00791	0.00641	0.00595	0.00589	0.00589	0.00589
251	SG881101	MSNA		N33	0.00693	0.00264	0.00612	0.01509	0.01682	0.02209	0.02265	0.00729	0.00969	0.00766	0.00723	0.00728	0.00728	0.00728
252	SG881101	MSNA		VRT	0.00256	0.00142	0.00289	0.00500	0.00552	0.00739	0.00841	0.00729	0.00453	0.00375	0.00295	0.00285	0.00285	0.00285
253	SG881101	LYON		N21	0.00278	0.00121	0.00269	0.00527	0.00537	0.00656	0.00738	0.00610	0.00553	0.00365	0.00317	0.00297	0.00297	0.00297
254	SG881101	LYON		N30	0.00357	0.00228	0.00386	0.01058	0.00881	0.00819	0.00682	0.00748	0.00650	0.00492	0.00422	0.00430	0.00430	0.00430
255	SG881101	LYON		VRT	0.00398	0.00149	0.00431	0.01185	0.01397	0.00782	0.00547	0.00550	0.00464	0.00454	0.00495	0.00483	0.00483	0.00483
256	SG881101	EMME		N33	0.00130	0.00142	0.00178	0.00252	0.00301	0.00345	0.00316	0.00326	0.00361	0.00247	0.00173	0.00178	0.00178	0.00178
257	SG881101	EMME		N65	0.00215	0.00275	0.00352	0.00527	0.00537	0.00457	0.00379	0.00387	0.00279	0.00282	0.00250	0.00230	0.00230	0.00230
258	SG881101	EMME		N32	0.00178	0.00207	0.00326	0.00325	0.00346	0.00225	0.00214	0.00280	0.00211	0.00191	0.00202	0.00225	0.00225	0.00225
259	SG881101	MIME		N56	0.00117	0.00039	0.00054	0.00107	0.00109	0.00130	0.00182	0.00154	0.00107	0.00155	0.00144	0.00189	0.00189	0.00189
260	SG881101	MIME		N32	0.00118	0.00047	0.00073	0.00096	0.00101	0.00135	0.00132	0.00169	0.00150	0.00174	0.00136	0.00155	0.00155	0.00155
261	SG881101	MIME		VRT	0.00121	0.00036	0.00062	0.00092	0.00103	0.00130	0.00108	0.00145	0.00111	0.00123	0.00153	0.00163	0.00163	0.00163
262	SG881101	ISFL		N17	0.00538	0.00552	0.00842	0.01202	0.01212	0.01146	0.01027	0.00731	0.00741	0.00678	0.00557	0.00553	0.00553	0.00553
263	SG881101	ISFL		N26	0.00551	0.00375	0.01116	0.01128	0.01245	0.01573	0.00913	0.00855	0.00909	0.01071	0.00738	0.00596	0.00596	0.00596
264	SG881101	ISFL		VRT	0.00401	0.00253	0.00723	0.00710	0.01141	0.00982	0.00861	0.00576	0.00522	0.00490	0.00486	0.00439	0.00439	0.00439
265	SG881101	CKY		N00	0.06311	0.01633	0.02410	0.08517	0.12301	0.23982	0.13103	0.10481	0.07164	0.06511	0.06546	0.06360	0.06360	0.06360
266	SG881101	CKY		N90	0.09152	0.02281	0.03821	0.05264	0.10005	0.44063	0.29607	0.15159	0.11678	0.09611	0.09660	0.09235	0.09235	0.09235
267	SG881101	CKY		VRT	0.03297	0.01623	0.03552	0.08274	0.08650	0.10586	0.09157	0.06292	0.04259	0.03412	0.03738	0.03443	0.03443	0.03443
268	SG881101	GSC1		000	0.12109	0.00715	0.01299	0.05625	0.11734	0.19560	0.30121	0.35807	0.16950	0.16035	0.15124	0.13197	0.13197	0.13197
269	SG881101	GSC1		270	0.09729	0.00557	0.01443	0.03843	0.08112	0.22387	0.28026	0.25192	0.15742	0.13017	0.12102	0.10958	0.10958	0.10958
270	SG881101	GSC1		VRT	0.06240	0.01116	0.02242	0.04081	0.09402	0.13525	0.16654	0.13050	0.16565	0.08432	0.08633	0.07501	0.07501	0.07501
271	SG881101	GSC2		051	0.05057	0.01620	0.01220	0.04389	0.04650	0.09727	0.11673	0.11947	0.07995	0.05627	0.05196	0.05292	0.05292	0.05292
272	SG881101	GSC2		321	0.05064	0.01797	0.02284	0.05162	0.13495	0.14209	0.14955	0.11007	0.11139	0.05995	0.05093	0.05655	0.05655	0.05655
273	SG881101	GSC2		VRT	0.01987	0.01508	0.02357	0.02436	0.05902	0.04348	0.06153	0.04470	0.03298	0.02358	0.02339	0.02494	0.02494	0.02494
274	SG881101	GSC5		097	0.02688	0.00363	0.01113	0.02400	0.01996	0.03666	0.05343	0.06254	0.09050	0.06587	0.06144	0.03797	0.03797	0.03797
275	SG881101	GSC5		007	0.00218	0.00119	0.00182	0.00145	0.00136	0.00155	0.00157	0.00189	0.00201	0.00234	0.00326	0.00610	0.00610	0.00610
276	SG881101	GSC5		VRT	0.05328	0.01149	0.03235	0.03384	0.02747	0.06295	0.06260	0.09346	0.11085	0.11867	0.06593	0.07129	0.07129	0.07129
277	SG881101	GSC7		175	0.12522	0.05251	0.06599	0.14471	0.25225	0.40900	0.36019	0.28388	0.16318	0.12740	0.12956	0.13172	0.13172	0.13172
278	SG881101	GSC7		085	0.17392	0.03786	0.08598	0.23551	0.39200	0.56955	0.43795	0.35694	0.20454	0.18415	0.19730	0.20081	0.20081	0.20081
279	SG881101	GSC7		VRT	0.11486	0.02723	0.05148	0.10320	0.09682	0.16652	0.31990	0.50991	0.30701	0.15782	0.16559	0.14783	0.14783	0.14783
280	SG881101	GSC8		063	0.12418	0.03373	0.07968	0.13460	0.14678	0.26487	0.28624	0.24726	0.19061	0.15213	0.14541	0.15302	0.15302	0.15302

Table 2.A-4 (continued)

Five-Percent Damped Spectral Accelerations (in g) for ENA Ground Motion Records

Rec	EQID	SI	NO	COMP	AMAX	F1	F1.5	F2.5	F3.45	F5	F7.5	F10	F15	F25	F34	F50	F75	F100
281	SG881101	GSC8	333		0.05987	0.01207	0.01823	0.04451	0.07607	0.10106	0.11916	0.10266	0.10794	0.10569	0.08664	0.06692		
282	SG881101	GSC8	VRT		0.06780	0.00832	0.02027	0.04335	0.14511	0.11919	0.28886	0.15015	0.09982	0.08215	0.07974	0.06923		
283	SG881101	GSC9	000		0.04634	0.02724	0.05609	0.05870	0.11434	0.10290	0.14019	0.08269	0.08605	0.06788	0.05210	0.05953		
284	SG881101	GSC9	270		0.05576	0.01719	0.03961	0.07223	0.10341	0.17978	0.22080	0.14121	0.07736	0.07114	0.05988	0.05994		
285	SG881101	GSC9	VRT		0.03661	0.00965	0.02906	0.07354	0.07863	0.10105	0.27359	0.05084	0.04453	0.03750	0.03988	0.04030		
286	SG881101	GSC10	000		0.04036	0.02536	0.03376	0.05401	0.06366	0.09418	0.14982	0.10973	0.08102	0.04894	0.04368	0.04129		
287	SG881101	GSC10	270		0.05700	0.02685	0.05929	0.11554	0.08727	0.09522	0.16406	0.16052	0.08789	0.06334	0.06243	0.06054		
288	SG881101	GSC10	VRT		0.02329	0.00930	0.02685	0.04130	0.04040	0.05304	0.07295	0.05314	0.06005	0.04261	0.02963	0.02669		
289	SG881101	GSC14	000		0.01384	0.00341	0.01104	0.02041	0.03311	0.05633	0.04100	0.03198	0.02991	0.02125	0.01805	0.01697		
290	SG881101	GSC14	270		0.02328	0.00892	0.01081	0.02865	0.05687	0.04539	0.03517	0.03555	0.03272	0.03104	0.02881	0.02209		
291	SG881101	GSC14	VRT		0.02101	0.01081	0.01232	0.03047	0.06089	0.04539	0.03517	0.03555	0.03272	0.03104	0.02881	0.02209		
292	SG881101	GSC16	214		0.10650	0.00521	0.01417	0.03819	0.05816	0.10566	0.24342	0.17153	0.32293	0.23080	0.24563	0.17275		
293	SG881101	GSC16	124		0.13115	0.00912	0.05337	0.08879	0.08930	0.14696	0.17824	0.27240	0.21981	0.30688	0.37116	0.24553		
294	SG881101	GSC16	VRT		0.10245	0.00842	0.02429	0.05380	0.11734	0.19533	0.12954	0.17837	0.17585	0.23432	0.18207	0.17833		
295	SG881101	GSC17	000		0.15588	0.00438	0.01410	0.05348	0.06079	0.11919	0.12358	0.15482	0.36774	0.40242	0.32156	0.35554		
296	SG881101	GSC17	270		0.09109	0.00411	0.00810	0.02035	0.03928	0.10826	0.14787	0.19066	0.22506	0.28120	0.26078	0.21601		
297	SG881101	GSC17	VRT		0.04522	0.00278	0.00444	0.01632	0.04601	0.05455	0.10786	0.13527	0.16181	0.11008	0.09777	0.07231		
298	SG881101	GSC20	000		0.12545	0.01934	0.04022	0.13470	0.28656	0.22527	0.29771	0.24654	0.21523	0.20106	0.20359	0.14399		
299	SG881101	GSC20	270		0.10221	0.01713	0.03694	0.08726	0.14714	0.10391	0.25837	0.19654	0.25526	0.28079	0.20933	0.19257		
300	SG881101	GSC20	VRT		0.23434	0.01378	0.03423	0.09179	0.28888	0.31789	0.27360	0.25816	0.26492	1.00455	0.74995	0.37931		
301	SG881101	SBQ	Z		0.00546	0.01179	0.00979	0.00615	0.00581	0.00601	0.00577	0.00551						
302	SG881101	GSQ	Z		0.00326	0.00570	0.00462	0.00349	0.00341	0.00334	0.00339	0.00333						
303	SG881101	TRQ	Z		0.00438	0.00957	0.00908	0.00498	0.00468	0.00503	0.00492	0.00459						
304	SG881101	KLN	Z		0.00161	0.00354	0.00291	0.00164	0.00163	0.00164	0.00163	0.00307						
305	SG881101	GRQ	Z		0.00299	0.00594	0.00430	0.00328	0.00319	0.00310	0.00313	0.00307						
306	SG881101	WBO	Z		0.00283	0.00562	0.00351	0.00291	0.00286	0.00303	0.00291	0.00284						
307	SG881101	GGN	Z		0.00197	0.00255	0.00225	0.00200	0.00197	0.00202	0.00200	0.00197						
308	NH881200	HNBK	N90		0.00163	0.00013	0.00019	0.00060	0.00154	0.00238	0.00329	0.00371	0.00359	0.00424	0.00237	0.00177		
309	NH881200	HNBK	N00		0.00145	0.00013	0.00017	0.00063	0.00165	0.00315	0.00305	0.00295	0.00347	0.00352	0.00197	0.00158		
310	NH881200	HNBK	VRT		0.00169	0.00019	0.00013	0.00026	0.00099	0.00135	0.00280	0.00482	0.00402	0.00327	0.00200	0.00242		
311	NH881200	RIDG	N97		0.01276	0.00039	0.00067	0.00195	0.00444	0.01069	0.02828	0.03220	0.03232	0.02902	0.01589	0.01408		
312	NH881200	RIDG	N07		0.00996	0.00027	0.00068	0.00235	0.00391	0.01006	0.01380	0.02142	0.02687	0.02836	0.01499	0.01121		
313	NH881200	RIDG	VRT		0.00887	0.00015	0.00023	0.00042	0.00094	0.00250	0.00829	0.02133	0.02828	0.01863	0.01382	0.01142		
314	NH890400	HNBK	N90		0.00505	0.00046	0.00080	0.00190	0.00478	0.00810	0.00781	0.01316	0.01312	0.01081	0.01043	0.00582		
315	NH890400	HNBK	N00		0.00472	0.00041	0.00089	0.00201	0.00390	0.00697	0.00846	0.01288	0.01279	0.01617	0.00883	0.00565		
316	NH890400	HNBK	VRT		0.00346	0.00027	0.00037	0.00085	0.00240	0.00558	0.00668	0.01672	0.00988	0.00789	0.00621	0.00525		
317	NH890400	RIDG	N97		0.00861	0.00147	0.00290	0.00379	0.00843	0.00763	0.01740	0.02666	0.01793	0.01859	0.01200	0.00959		
318	NH890400	RIDG	N07		0.00733	0.00098	0.00181	0.00320	0.00730	0.01061	0.01738	0.02711	0.01756	0.01757	0.00929	0.00862		
319	NH890400	RIDG	VRT		0.01086	0.00031	0.00085	0.00106	0.00230	0.00508	0.01105	0.01696	0.03534	0.02362	0.03150	0.01386		
320	NH890400	OLAP	N70		0.00370	0.00024	0.00030	0.00044	0.00055	0.00100	0.00314	0.00749	0.01202	0.00676	0.00485	0.00426		
321	NH890400	OLAP	N34		0.00278	0.00025	0.00039	0.00048	0.00067	0.00123	0.00282	0.00594	0.01935	0.00909	0.00440	0.00356		
322	NH890400	OLAP	VRT		0.00347	0.00023	0.00031	0.00039	0.00072	0.00113	0.00299	0.00543	0.01935	0.00909	0.00550	0.00466		
323	MS890700	MSNA	335		0.00328	0.00019	0.00020	0.00022	0.00037	0.00152	0.00299	0.00539	0.00603	0.00807	0.01201	0.00907		
324	MS890700	MSNA	245		0.00579	0.00046	0.00019	0.00023	0.00031	0.00084	0.00257	0.00309	0.00515	0.01142	0.02796	0.01863		
325	MS890700	MSNA	VRT		0.00530	0.00016	0.00017	0.00023	0.00030	0.00038	0.00097	0.00210	0.00544	0.01088	0.01626	0.01673		

Table 2.A-4 (continued)

Five-Percent Damped Spectral Accelerations (in g) for ENA Ground Motion Records

REC	EQID	ST_NO	COMP	AMAX	F1	F1.5	F2.5	F3.45	F5	F7.5	F10	F15	F25	F34	F50	F75	F100
326	A900421	A16	E	0.00098	0.00003	0.00005	0.00012	0.00030	0.00072	0.00168	0.00357						
327	A900421	A16	N	0.00191	0.00006	0.00011	0.00030	0.00060	0.00137	0.00285	0.00482						
328	A900421	A16	Z	0.00047	0.00001	0.00002	0.00007	0.00012	0.00033	0.00083	0.00191						
329	A900421	A61	E	0.00217	0.00004	0.00009	0.00030	0.00055	0.00155	0.00351	0.00668						
330	A900421	A61	N	0.00139	0.00002	0.00005	0.00012	0.00023	0.00074	0.00250	0.00420						
331	A900421	A61	Z	0.00067	0.00001	0.00003	0.00010	0.00021	0.00038	0.00147	0.00217						
332	A900421	A54	E	0.00024	0.00001	0.00001	0.00005	0.00010	0.00026	0.00058	0.00087						
333	A900421	A54	N	0.00032	0.00001	0.00002	0.00009	0.00018	0.00046	0.00088	0.00104						
334	A900421	A54	Z	0.00044	0.00001	0.00002	0.00006	0.00011	0.00030	0.00111	0.00138						
335	A900421	A21	E	0.00061	0.00001	0.00003	0.00013	0.00032	0.00043	0.00126	0.00176						
336	A900421	A21	N	0.00047	0.00002	0.00004	0.00016	0.00031	0.00037	0.00143	0.00159						
337	A900421	A21	Z	0.00027	0.00001	0.00002	0.00005	0.00011	0.00044	0.00114	0.00088						
338	A900421	A64	E	0.00045	0.00001	0.00004	0.00013	0.00027	0.00065	0.00100	0.00070						
339	A900421	A64	N	0.00024	0.00001	0.00003	0.00006	0.00016	0.00024	0.00056	0.00052						
340	A900421	A64	Z	0.00038	0.00001	0.00001	0.00004	0.00009	0.00034	0.00083	0.00073						
341	A900421	A11	E	0.00020	0.00001	0.00001	0.00004	0.00009	0.00017	0.00031	0.00046						
342	A900421	A11	N	0.00012	0.00000	0.00001	0.00003	0.00005	0.00014	0.00027	0.00044						
343	A900421	A11	Z	0.00008	0.00000	0.00001	0.00003	0.00005	0.00008	0.00018	0.00023						
344	CG900900	EBN	Z	0.00003	0.00000	0.00000	0.00001	0.00002	0.00004	0.00008	0.00015						
345	CG900900	OLAP	340	0.02071	0.00038	0.00065	0.00286	0.00375	0.00554	0.01483	0.02959	0.06205	0.05788	0.03526	0.02801		
346	CG900900	OLAP	070	0.01899	0.00031	0.00053	0.00110	0.00174	0.00461	0.01064	0.03366	0.05797	0.05143	0.04117	0.02695		
347	CG900900	OLAP	VRT	0.03414	0.00035	0.00054	0.00152	0.00275	0.00444	0.00954	0.03130	0.09866	0.16245	0.07859	0.04426		
348	CG900900	RIDG	007	0.00542	0.00050	0.00137	0.00238	0.00396	0.00603	0.01530	0.01643	0.01268	0.00830	0.00632	0.00587		
349	CG900900	RIDG	097	0.00515	0.00039	0.00121	0.00201	0.00316	0.00737	0.01297	0.01889	0.01600	0.01086	0.00662	0.00579		
350	CG900900	RIDG	VRT	0.00409	0.00019	0.00056	0.00087	0.00134	0.00240	0.00528	0.00875	0.01539	0.00705	0.00730	0.00466		
351	CG900900	RSCO	000	0.00745	0.00075	0.00193	0.00307	0.00863	0.01406	0.02850	0.02543	0.01474	0.00933	0.00764	0.00772		
352	CG900900	RSCO	090	0.01494	0.00088	0.00177	0.00669	0.01105	0.02652	0.04238	0.03652	0.02088	0.01652	0.01668	0.01531		
353	CG900900	RSCO	VRT	0.00607	0.00029	0.00056	0.00126	0.00215	0.00608	0.00992	0.01645	0.02087	0.01868	0.00951	0.00676		
354	ML901000	MSNA	245	0.00560	0.00047	0.00130	0.00465	0.00675	0.01533	0.02126	0.01561	0.00880	0.00610	0.00581	0.00555		
355	ML901000	MSNA	335	0.00307	0.00022	0.00038	0.00274	0.00297	0.00556	0.01145	0.01160	0.00675	0.00439	0.00365	0.00349		
356	ML901000	MSNA	VRT	0.00197	0.00021	0.00047	0.00104	0.00225	0.00491	0.00515	0.00556	0.00610	0.00483	0.00275	0.00239		
357	ML901000	GRQ	Z	0.09164	0.23522	0.22977	0.10869	0.10366	0.11092	0.10399	0.09761						
358	ML901000	TRQ	Z	0.02936	0.07323	0.06111	0.03285	0.03098	0.03620	0.03272	0.03047						
359	ML901000	OTT	Z	0.00140	0.00493	0.00349	0.00149	0.00141	0.00164	0.00153	0.00141						
360	ML901000	CKO	Z	0.01101	0.02254	0.01813	0.01179	0.01095	0.01245	0.01192	0.01114						
361	ML901000	WBO	Z	0.01004	0.02155	0.01424	0.01059	0.00989	0.00992	0.01038	0.00998						
362	ML901000	MNT	Z	0.00105	0.00439	0.00197	0.00113	0.00107	0.00125	0.00111	0.00106						
363	ML901000	DPQ	Z	0.00206	0.00541	0.00539	0.00222	0.00201	0.00235	0.00222	0.00204						
364	ML901000	EEO	Z	0.00042	0.00136	0.00078	0.00047	0.00044	0.00050	0.00046	0.00044						
365	ML901000	SBO	Z	0.00044	0.00113	0.00069	0.00045	0.00044	0.00046	0.00045	0.00044						
366	ML901000	WEO	Z	0.00093	0.00322	0.00213	0.00108	0.00101	0.00105	0.00102	0.00097						
367	ML901000	A54	E	0.00029	0.00006	0.00018	0.00065	0.00075	0.00122	0.00080	0.00091						
368	ML901000	A54	N	0.00065	0.00011	0.00018	0.00099	0.00106	0.00163	0.00136	0.00102						
369	ML901000	A54	Z	0.00040	0.00010	0.00025	0.00081	0.00068	0.00123	0.00081	0.00101						
370	ML901000	SNO	Z	0.00015	0.00037	0.00027	0.00016	0.00015	0.00016	0.00016	0.00015						
371	ML901000	SNO	Z	0.00011	0.00031	0.00017	0.00012	0.00011	0.00012	0.00012	0.00011						

Table 2.A-4 (continued)

Five-Percent Damped Spectral Accelerations (in g) for ENA Ground Motion Records

Rec	EQID	SI_NO	COMP	AMAX	F1	F1.5	F2.5	F3.45	F5	F7.5	F10	F15	F25	F34	F50	F75	F100
372	ML901000	A11	E	0.00025	0.00012	0.00022	0.00057	0.00071	0.00071	0.00060	0.00063						
373	ML901000	A11	N	0.00018	0.00016	0.00025	0.00053	0.00064	0.00067	0.00070	0.00037						
374	ML901000	A11	Z	0.00026	0.00012	0.00030	0.00064	0.00075	0.00068	0.00046	0.00045						
375	ML901000	A16	E	0.00024	0.00009	0.00018	0.00042	0.00070	0.00065	0.00064	0.00052						
376	ML901000	A16	N	0.00022	0.00010	0.00015	0.00041	0.00042	0.00073	0.00051	0.00045						
377	ML901000	A16	Z	0.00028	0.00011	0.00020	0.00035	0.00076	0.00115	0.00068	0.00061						
378	ML901000	A61	E	0.00029	0.00006	0.00016	0.00047	0.00063	0.00069	0.00080	0.00072						
379	ML901000	A61	N	0.00033	0.00014	0.00018	0.00050	0.00090	0.00102	0.00087	0.00070						
380	ML901000	A61	Z	0.00015	0.00005	0.00014	0.00036	0.00046	0.00053	0.00044	0.00036						
381	ML901000	S20	Z	0.00011	0.00041	0.00019	0.00012	0.00012	0.00012	0.00012	0.00011						
382	ML901000	A64	E	0.00023	0.00005	0.00014	0.00034	0.00063	0.00059	0.00077	0.00056						
383	ML901000	A64	N	0.00024	0.00008	0.00015	0.00054	0.00073	0.00069	0.00070	0.00051						
384	ML901000	A64	Z	0.00016	0.00007	0.00018	0.00032	0.00038	0.00058	0.00045	0.00034						
385	ML901000	A21	E	0.00040	0.00010	0.00022	0.00076	0.00103	0.00185	0.00106	0.00079						
386	ML901000	A21	N	0.00522	0.00087	0.00163	0.00382	0.00433	0.00906	0.01011	0.00926						
387	ML901000	A21	Z	0.00024	0.00008	0.00018	0.00052	0.00052	0.00092	0.00074	0.00052						
388	NM901100	R1DG	007	0.00567	0.00026	0.00046	0.00155	0.00380	0.00812	0.01737	0.02112	0.01338	0.00866	0.00702	0.00608		
389	NM901100	R1DG	097	0.00781	0.00032	0.00087	0.00263	0.00671	0.01342	0.01516	0.02631	0.01610	0.01138	0.00913	0.00842		
390	NM901100	R1DG	VRT	0.00354	0.00017	0.00017	0.00036	0.00092	0.00183	0.00558	0.00943	0.01919	0.00917	0.00538	0.00429		
391	SQ901200	HERB	000	0.00651	0.00028	0.00035	0.00130	0.00174	0.00285	0.00401	0.00943	0.01789	0.02356	0.02060	0.01049		
392	SQ901200	HERB	090	0.00743	0.00038	0.00028	0.00114	0.00276	0.00280	0.00637	0.00895	0.02224	0.02272	0.01756	0.01104		
393	SQ901200	HERB	VRT	0.00461	0.00030	0.00025	0.00049	0.00117	0.00207	0.00390	0.00729	0.01826	0.01853	0.01587	0.00683		
394	NM910500	RSCO	000	0.00304	0.00007	0.00017	0.00046	0.00108	0.00170	0.00718	0.01281	0.00600	0.00449	0.00336	0.00320		
395	NM910500	RSCO	090	0.00220	0.00007	0.00007	0.00015	0.00027	0.00073	0.00244	0.00685	0.00483	0.00417	0.00285	0.00236		
396	NM910500	RSCO	VRT	0.00245	0.00010	0.00007	0.00021	0.00043	0.00106	0.00391	0.00694	0.00851	0.00496	0.00309	0.00307		
397	NM910501	OLAP	340	0.00796	0.00043	0.00082	0.00217	0.00208	0.00285	0.00506	0.01569	0.03225	0.01596	0.01121	0.00865		
398	NM910501	OLAP	070	0.00698	0.00029	0.00040	0.00088	0.00164	0.00239	0.00417	0.01550	0.03678	0.01693	0.01085	0.00967		
399	NM910501	OLAP	VRT	0.00880	0.00025	0.00046	0.00086	0.00116	0.00146	0.00408	0.00843	0.04862	0.02368	0.01336	0.01021		
400	NM910501	R1DG	007	0.00369	0.00078	0.00176	0.00307	0.00455	0.00645	0.01191	0.00949	0.00879	0.00573	0.00393	0.00402		
401	NM910501	R1DG	097	0.00524	0.00090	0.00190	0.00259	0.00331	0.00944	0.01223	0.01415	0.02275	0.00974	0.00656	0.00594		
402	NM910501	R1DG	VRT	0.00642	0.00035	0.00094	0.00132	0.00159	0.00269	0.01174	0.01595	0.02229	0.01063	0.00907	0.00726		
403	NM910501	RSCO	000	0.12723	0.01688	0.04279	0.07582	0.08182	0.14642	0.36161	0.39451	0.19249	0.15104	0.00907	0.13310		
404	NM910501	RSCO	090	0.09827	0.00401	0.01019	0.02143	0.05354	0.18377	0.38195	0.37284	0.18841	0.15701	0.11333	0.10499		
405	NM910501	RSCO	VRT	0.10840	0.00275	0.00547	0.01169	0.02491	0.06798	0.25822	0.45797	0.25842	0.26714	0.13873	0.12494		
406	NM910501	CHTN	000	0.00167	0.00032	0.00114	0.00149	0.00245	0.00383	0.00607	0.00614	0.00381	0.00254	0.00205	0.00191		
407	NM910501	CHTN	090	0.00168	0.00023	0.00110	0.00174	0.00283	0.00471	0.00618	0.00649	0.00448	0.00260	0.00199	0.00179		
408	NM910501	CHTN	VRT	0.00144	0.00033	0.00048	0.00073	0.00110	0.00152	0.00279	0.00377	0.00319	0.00471	0.00358	0.00194		
409	NM910501	FTPL	310	0.00751	0.00394	0.00995	0.01218	0.01793	0.02143	0.02405	0.01521	0.00930	0.00837	0.00799	0.00800		
410	NM910501	FTPL	040	0.00741	0.00165	0.00464	0.01011	0.01818	0.02764	0.01613	0.01078	0.01322	0.00994	0.00858	0.00768		
411	NM910501	FTPL	VRT	0.00557	0.00071	0.00209	0.00618	0.01454	0.02113	0.01551	0.01600	0.00988	0.00745	0.00665	0.00583		

Table 2.A-5

Site Geology for Recording Stations

Eastern U.S. Strong Ground Motion Stations

Maine

All stations in Maine, which were installed by NCEER in 1987 and 1988, are listed as bedrock sites. The thickness of unconsolidated materials listed for each site represents the maximum or average thickness of these deposits observed in the vicinity of the station.

East Machias (EMME)

44.739°N 67.489°W

Devonian plutonic rocks, including granite, granodiorite, and quartz monzonite underlie the site. Quaternary deposits, consisting of glacial-marine silts and clays (Presumpscot Formation) of unknown thickness (probably less than 10–20 meters), are mapped throughout the region (scale 1:24,000, Borns, 1974; scale 1:62,500, Gates, 1981); however, the station is located on granitic bedrock (Busby, 1990). Region of low relief. Confidence: Moderate to High.

Milo (MIME)

45.244°N 69.043°W

Silurian Sangerville Formation, including calcareous quartzite and phyllite, underlies the site. Devonian quartz monzonites are exposed 19 km to the north, and may be present at depth below the station. Region is covered with a thin veneer of glacial till (sand, silt, and clay, thickness of a few meters?) (scale 1:62,500, Griffin, 1971a,b,c,d; Hanson and Caldwell, 1981). Station located on the side of a moderately sloping hill above a narrow valley. Confidence: High.

Island Falls (ISFL)

46.031°N 68.206°W

Devonian Pleasant Lake pluton, comprised of granodiorite and quartz monzonite, underlies the site. Metamorphosed sedimentary rocks overlie the pluton 1 km to the north. Region is covered with a thin veneer of glacial till (sand, silt, and clay, thickness of a few meters?) (scale 1:62,500, Pavlides, 1972, and Brewer and Newman, 1986; scale 1:110,000, Pavlides and Canney, 1964). Station is located on the side of a moderately to steeply sloping hill. Confidence: High.

Dickey (DCKY)

47.101°N 69.082°W

Devonian Seboonook Formation, including slate and graywacke (metasedimentary pelite), underlies the station site on the St John River floodplain. Quaternary floodplain and stream terrace deposits (gravel, sand, and silt) of unknown thickness (probably less than 10–20 meters?) overlie bedrock along the river; however, the station is reported to be located on bedrock (Busby, 1990). The Seboonook Formation is believed to be 600 meters thick (scale 1:62,500, Roy, 1982, and Lowell, 1986). Station is located in a narrow valley below moderate to steeply sloping hills. Confidence: Moderate to High.

Mississippi

Arkabutla Dam, leftcrest 2512 (2444A)

34.760°N 90.120°W

left toe 2513 (2444B)

right abutment 2514 (2444C)

Eocene Kosciusko Formation (Claiborne Group) mapped at station site. The Kosciusko Formation consists of irregularly bedded sand and clay with some quartzite, is weakly to moderately consolidated, and is about 120 meters thick. The underlying Claiborne sediments include the Basic City Shale and Meridian Sand (each unit approximately 60 to 90 meters thick) of the Tallahata Formation. The underlying Eocene Wilcox Group (shale, silt, and sand) and Paleocene Porters Hill Clay have a total thickness of approximately 260 meters. The tertiary rocks unconformably overlie Paleozoic metasedimentary rocks at a depth of approximately 550 meters (scale 1:500,000, Brown, 1947, and Bickers, 1969). The region is mantled with approximately 30 meters of Pleistocene to Holocene loess; however, the Coldwater River may be incised through most of the Quaternary deposits at the dam site. Unconsolidated (Quaternary) materials in the river bed may have been removed during dam construction. Region of low relief. Structure is an earthfill dam, 29 meters high. Confidence: High.

Missouri

Fort Pillow (FTPL)

36.945°N 89.392°W

Charleston (CHTN)

Quaternary alluvial deposits in Mississippi Embayment mapped at station site. These deposits include clay, silt, sand, and gravel (35 meters thick), overlying: Paleocene Wilcox Group (Memphis Sand, weakly consolidated with some clay shale, 94 meters thick), Paleocene Midway Group (Porters Creek Clay, 60 meters thick), Cretaceous McNairy Sand (with clay shale, 30 meters thick), and Paleozoic rocks (limestone?, unconformity at a depth of 220 meters) (scale 1:400,000, Grohskopf, 1955; 1:500,000; Anderson, 1979; 1:1,000,000, Mesko, 1988). Region of low relief. Confidence: High.

New Madrid, Noranda Aluminum Plant (2420)

36.510°N 89.570°W

Holocene and Pleistocene alluvium of the Mississippi Embayment (silt, clay, sand, and gravel, 52 meters thick) mapped at station site. Alluvium overlies: Eocene Claiborne Group, including Cockfield Formation (sand, 30 meters thick), Cook Mountain Formation (silt and clay, 24 meters thick), and Memphis Sand (137 meters thick); Paleocene Wilcox Group, includes Flour Island Formation (silt and clay, 30 meters thick) and Fort Pillow Sand (30 meters thick); Paleocene Midway Group (Porters Creek Clay, 90 meters thick); Cretaceous McNairy Sand (105 meters thick); Ordovician Jefferson City Dolomite (unconformity at a depth of 500 meters) (scale 1:400,000, Grohskopf, 1955; 1:500,000; Anderson, 1979; 1:1,000,000, Mesko, 1988). Station is located on the west bank of the Mississippi River. Region of low relief. Confidence: High.

Wappapello Dam, spillway (2415B)

36.930°N 90.270°W

right toe (2417)

right crest (2415A)

Ordovician Roubidoux Formation (shale and sandstone 32 meters thick mapped at station site. Roubidoux Formation overlies sandstone, 44 meters thick), which overlies Gasconade Dolomite (minimum thickness of 44 meters). Based on log of water well at dam, Paleozoic bedrock is exposed at the surface in the river valley (scale 1:400,000, Grohskopf, 1955; scale 1:500,000; Anderson, 1979). The stations are located on or below a 33-meter high earth and rock dam. Stations located in broad valley(?). Confidence: High.

Risco, NCEER (RSCO)

36.550°N 89.774°W

Quaternary alluvial deposits of the Mississippi Embayment (sand, silt, and gravel, 40 meters thick) mapped at station site. Alluvial deposits overlie: Eocene Claiborne Group, includes Cook Mountain Formation (silt and clay, 33 meters thick) and Memphis Sand (140 meters thick); Paleocene Wilcox Group, includes Flour Island Formation (silt and clay, approximately 30 meters thick) and Fort Pillow Sand (24 meters thick); Paleocene Midway Group (Porters Creek Clay, 105 meters thick); Cretaceous McNairy Sand (97 meters thick); Ordovician Jefferson City Dolomite (unconformity at a depth of 470 meters) (scale 1:400,000, Grohskopf, 1955; scale 1:500,000; Anderson, 1979; scale 1:1,000,000, Mesko, 1988). The sediments may be locally overlain by loess in this area. Region of low relief. Confidence: High.

Old Appleton (OLAP)

37.572°N 89.687°W

Ordovician Platin Limestone (with shale and chert, 105 to 120 meters thick) mapped at station site. Limestone overlies: Pecatonica Formation (limestone, dolomite, and shale, 43 to 52 meters thick), Joachin Dolomite (52 to 59 meters thick), St. Peter Sandstone (more than 45 meters thick) (scale 1:24,000; Amos, 1985; scale 1:500,000; Anderson, 1979). Station site has less than 2 meters of soil overlying bedrock (Busby, 1990). Detailed map for quadrangle located 3.5 miles north of the station. Region of low or moderate relief. Confidence: High.

New Hampshire**Franklin Falls Dam (2627)**

43.447°N 71.660°W

Devonian micaceous quartzite, mica schist, and gneiss mapped at station site (scale 1:250,000, Billings, 1956). Station located in a narrow valley. Confidence: Moderate. Site conditions for three instruments described below.

downstream (2627A) Fluvial/alluvial deposits reported to be 30 meters thick.

abutment (2627B) Boulder fill, 3 meters thick.

crest (2627C) Structure is an earthfill dam, 43 meters high. Unconsolidated glacial/fluvial deposits in channel may have been removed during dam construction.

New York**Palisades (PAL)**

41.004°N 73.909°W

Triassic Palisade Diabase (sill) underlies station site. Sill is underlain by Stockton and Brunswick formations; these units, which include sandstone, conglomerate, and mudstone, are exposed 0.4 km west of the station. The Hudson River lies 1 km east of the station. Older rocks exposed to the east include Cambrian-Ordovician Inwood marble, approximately 2.5 km east of the river, Upper Proterozoic Fordham Gneiss, 3.2 km east of the river, and Manhattan Formation schist and gneiss, approximately 5 km east of the river (scale 1:250,000, New York State Museum, 1970). Region of low or moderate relief. Confidence: Moderate.

Newcomb (NEWC)

43.971°N 74.224°W

Precambrian Grenville series crystalline limestone mapped at station site, overlying Precambrian granite syenite. The granite is exposed approximately 150 meters to the south, and 0.8 km to the north of the station. The limestone/marble is interpreted to be a linear body, 0.8 km in width, 4 km or more in length, and extending to a depth of 1 km or more (scale 1:62,500, Balk, 1932). Station located in a narrow valley. Confidence: High.

Tupper Lake (JORD)

44.378°N 74.605°W

Precambrian hornblende granite with amphibolite mapped at station site, and granitic gneiss is exposed 2 km to the southwest (scale 1:62,500, Buddington and Leonard, 1962, Buddington and others, 1954). Quaternary alluvial-fluvial-glacial(?) deposits at the station are 20 meters thick, overlying hornblende biotite gneiss (Busby, 1990) Station located in a narrow valley. Confidence: High.

Massena (MSNA)

44.998°N 74.847°W

Paleozoic-Middle Ordovician Chazy Group limestones mapped at station site. Other limestone formations are exposed to the north and south of the station (scale 1:250,000, New York State Museum, 1970). Station sits on 15 meters of silty clay overlying the Chazy Group (Busby, 1990). Station located on an island in the St. Lawrence River. Confidence: High. Instrument located in buried concrete valus, depth 2.3 m.

Lyon Mountain (LYON)

44.726°N 73.911°W

Precambrian granite-gneiss plutonic rocks underlie station site (scale 1:62,500, Postel, 1952). Station located in a narrow valley. Confidence: High.

Ohio**Perry Plant, bldg foundation (PPBF)**

41.800°N 81.142°W

Station located in building foundation, which is founded on Devonian shale (minimum thickness of 45 meters) (Glassmoyer and Borchardt, 1990). Paleozoic sedimentary rocks (shale and sandstone) underlie region to depths of several kilometers (?) Surficial deposits in region consist of Pleistocene (Wisconsinan) lacustrine sediments, including silt and fine sand (4.5 meters) overlying glacial till (13.8 meters) (based on borehole data from plant site) (scale 1:63,360, White, 1980; Glassmoyer and Borchardt, 1990). Devonian Olentangy and Ohio Formation (shale); Mississippian and Pennsylvanian shales, sandstones and limestones crop out 10 km to the southwest (scale 1:500,000, Geological Map of Ohio, Bownocker, reprinted 1981). Region of low relief. Confidence: High

South Carolina**Monticello Dam, center crest (2532)**

34.300°N 81.333°W

Carboniferous Winnsboro Plutonic Complex (granitic rocks) mapped at station site. Cambrian amphibolite and hornblende gneiss exposed 1 km to the south of the station (scale 1:71,000, Secor and others, 1982). Region of low relief. Confidence: High.

Tennessee**Hornbeak (HNBK)**

36.333°N 89.296°W

Pleistocene loess (informally the Loess Hills Bluffs) mapped at station site. Loess consists of silt, silty clay, and minor sand, as much as 30 meters thick along the plains that border the Mississippi alluvial plain (floodplain). The loess is underlain by Plio-Pleistocene gravels (0 to 4.5 meters thick) and Eocene Jackson Formation (clay, claystone, and sandstone, all weakly consolidated, thickness of 100 meters or more). The Jackson Formation unconformably overlies older Tertiary sedimentary units (Claiborne Formation and Wilcox Group, sand, silt, and clay, total thickness of approximately 450 meters, including Jackson Formation). Underlying units include Paleocene Midway Group (Porters Creek Shale, top at approximately 640 meters depth, 150 meters thick), Cretaceous McNairey Sand (depth?, 90 meters thick), and Cambrian Bonnetterre Formation (contact at depth of approximately 730 meters, dolomite, limestone, and shale) (scale 1:250,000; Hardeman, 1966; Blythe and others, 1975; scale 1:400,000, Grohskopf, 1955). Instrument is seated in a steel box partly buried in clay soil. Region of low relief. Confidence: Moderate to High.

Ridgely (RIDG) 36.264°N 89.481°W
Holocene and Pleistocene alluvial deposits in the Mississippi Embayment mapped at station site. The alluvium consists chiefly of clay, silt, sand, and gravel, commonly 30 to 45 meters thick, unconformably overlying middle Eocene Claiborne sediments (clay, silt, and sand, weakly consolidated). Underlying units include Paleocene Midway Group (Porters Creek Shale, top at 445 meters depth, 125 meters thick), Cretaceous McNairey Sand (depth?, 200 meters thick), and Cambrian Bonnetterre Formation(?) (contact at depth of 770 meters, dolomite, limestone, and shale) (scale 1:250,000; Hardeman, 1966; Blythe and others, 1975; scale 1:400,000, Grohskopf, 1955). Instrument is seated in partly buried steel box. Region of low relief. Confidence: Moderate to High.

Tiptonville (2446) 36.370°N 89.410°W
Holocene and Pleistocene alluvial deposits in the Mississippi Embayment mapped at station site. The alluvium consists chiefly of clay, silt, sand, and gravel, commonly 30 to 45 meters thick, unconformably overlying middle Eocene Claiborne sediments (clay, silt, and sand, weakly consolidated). Underlying units include Paleocene Midway Group (Porters Creek Shale, top at 425 meters depth, 120 meters thick), Cretaceous McNairey Sand (depth?, 105 meters thick), and Cambrian Bonnetterre Formation (contact at depth of 650 meters, dolomite, limestone, and shale), Cambrian Lamote Formation (? quartzite, minimum Paleozoic thickness of 515 meters, to total depth of 1215 meters). (scale 1:250,000; Hardeman, 1966; Blythe and others, 1975; scale 1:400,000, Grohskopf, 1955). Region of low relief. Confidence: Moderate to High.

Vermont

Union Village Dam (2632) 43.794°N 72.258°W
Devonian Gile Mountain Formation, including interbedded quartzite, phyllite, limestone, and micaceous schist, mapped at the station site. Slate and greenstone exposed in fault contact 0.2 and 0.6 km to the southeast, respectively. The greenstone is underlain by slate at shallow depths (less than 1 km) (scale 1:62,500, Hadley, 1950 and Doll, 1945). Station located in a narrow valley. Confidence: High. Site conditions for three instruments described below.

left abutment (2632B) Boulder fill, 2 meters thick.

downstream (2632C) Boulder fill, 2 meters thick, overlying 9 meters of glacial moraine.

crest (2632A) Structure is an earth gravity dam, 52 meters high. Unconsolidated materials (alluvium and glacial moraine) may have been removed during dam construction.

North Hartland Dam, abutment (2629A) 43.604°N 72.361°W
crest (2629B)
Ordovician-Devonian Gile Mountain Formation, including phyllite and mica schist, mapped at station site, and quartzite and quartz-mica schist exposed 0.2 km to the west. Chlorite schist and phyllite of the Oxfordville Formation are exposed 0.5 km to the east (scale 1:62,500, Lyons, 1955). Station located in a narrow valley. Structure in an earth dam, 37 meters high. Confidence: High.

White River Jct., VA Hospital (2604) 43.630°N 72.330°W
Ordovician Oxfordville Formation, including mica schist, hornblende schist, amphibolite, and feldspathic schist, mapped at station site. Gneiss and chlorite schist exposed above the Ammonoosuc thrust fault, 0.3 km to the west. Mica schist and quartzite of the Gile Mountain Formation are exposed across the Monroe fault, 1.9 km to the west (scale 1:62,500, Lyons, 1955). Station located in a narrow valley. Confidence: High.

Ball Mountain Dam, crest (2617) 43.120°N 72.780°W
Precambrian Mount Holly Complex, including biotite gneiss with some quartzite, underlies the station site. Cambrian Bully Hill gneiss is exposed 0.5 km to the east (scale 1:250,000, Vermont Geological Survey, 1961). Structure is an earth and rock dam, 81 meters high. Station located in a narrow valley. Confidence: Moderate.

**North Springfield Dam, crest (2630A)
downstream (2630B)**

43.338°N 72.511°W

Precambrian Mount Holly Complex, including biotite gneiss and schist, underlies the station site. Cambrian schist and gneiss exposed 0.8 km to the east (scale 1:250,000, Vermont Geological Survey, 1961). Station located in a narrow valley. Structure is an earth gravity dam, 57 meters high. Confidence: Moderate.

Quebec: Eastern Canada Strong Motion Network, Geological Survey of Canada

Comments on the instrument housing and site conditions for the ECSNM-GSC stations were provided by Phil Munro (Geological Survey of Canada).

St. Ferreol (GSC #1) (USGS 6209)

47.126°N 70.828°W

Precambrian gneiss underlies station site. Monzonites and migmatites crop out at distances greater than 5 km to the southeast and northeast (scale 1:250,000; Laurin and Sharma, 1975). Comments: Washed bedrock near a hydroelectric project. Confidence: Moderate to High. Instrument in buried concrete vault near power plant.

Quebec (GSC #2) (USGS 6208)

46.778°N 71.275°W

Paleozoic limestones and shales of St. Lawrence Lowlands mapped at station site. Sedimentary rocks may be several hundred meters thick or several kilometers thick. Underlain by Precambrian crystalline rocks (scale 1:250,000; Laurin and Sharma, 1975). Comments: Exact site conditions uncertain. Bedrock (type uncertain) crops out near the building. Three-story university building constructed in the 1950s above the St. Lawrence River. Foundation appears to be set in bedrock. Confidence: Low to Moderate.

Tadoussac (GSC #5) (USGS 6211)

48.143°N 69.719°W

Ordovician granite and gneiss underlie station site. Possibly overlain by thin veneer (less than a few tens of meters at most?) of Holocene lake, stream, or swamp deposits (or unconsolidated glacial drift). Site is located at contact between granite and Holocene deposits (scale 1:63,360; Miller, 1973). Comments: Site located in weathered bedrock. Building is on bluff, with 15–20 meters (?) of elevation above the river (elevation about 60 meters). Rugged terrain, with little soil cover. Station installed by National Research Council, on pier in crawl space. There is sand (fill) around the pier, but the pier probably is set on bedrock. Confidence: Moderate.

Baie St. Paul (GSC #7) (USGS 6212)

47.442°N 70.507°W

Quaternary fluvial/marine sediments, overlying Paleozoic St. Lawrence Lowlands sedimentary rocks (sandstone?), mapped at station site. Quaternary sediments appear to be more than 20 meters thick. Thickness of Paleozoic sedimentary rocks uncertain (hundreds of meters to several kilometers?); underlain by Precambrian granite/gneiss (scale 1:63,360; Rondot, 1969; scale 1:250,000; Laurin and Sharma, 1975; Lyons and others, 1980). Comments: Deep alluvium site in a narrow valley. Confidence: Moderate.

La Malbaie (GSC #8) (USGS 6203)

47.655°N 70.153°W

Quaternary marine/fluvial sediments (alluvium), or Paleozoic limestones and shales (St. Lawrence Lowlands rocks) mapped at station site. Sedimentary rocks near La Malbaie are less than 100 meters thick, and overlie Precambrian granite-gneiss-pegmatites (scale 1:250,000; Laurin and Sharma, 1975; Lyons and others, 1980). Comments: Site conditions uncertain, but listed as bedrock. No rock outcrops nearby. Pitch of street is 20°; thus the topography suggests that there is little overburden in this area, and that the building foundation is set in bedrock. Confidence: Low.

- St. Pascal (GSC #9) (USGS 6210)** 47.526°N 69.805°W
 Station located at contact between Ordovician Kamouraska Formation (quartzite plus limestone conglomerate) and Riviere Ouelle Formation (shales and siltstones). These rocks are tightly folded (modeled thickness of 7 km; Lyons and others, 1980), and overlie Precambrian crystalline rocks (scale 1:63,360; Hubert, 1973). Elevation 55 ±3 meters. Comments: Bedrock outcrop a few meters high located near building; topography indicates that the station is underlain by less than 1 to 5 meters of alluvium. Entire area is landscaped and paved. Confidence: High.
- Riviere Ouelle (GSC #10)** 47.476°N 69.996°W
 Tightly folded Cambrian St. Roch Formation (siltstone, mudstone, shale, arkose, limestone) mapped at station site. Thickness of sediments uncertain (possibly 7 km; Lyons and others, 1980), overlying Precambrian crystalline rocks. Ordovician shales, siltstones, arkose, and quartzite located to southeast of station (scale 1:63,360; Beland, 1957). Comments: Located on low-lying flat terrace (swampy area on 1973 map) along south bank of St. Lawrence River (elevation 12 ±3 meters). Station set on steel plate in steel culvert (shielded inside with concrete), set on concrete pad on washed bedrock. Confidence: High.
- St. Lucie de Beauregard (GSC #14)** 46.741°N 70.017°W
 Ordovician slates, sandstones, cherts, limestone, possibly some tuffs or other basic volcanic rocks mapped in area (scale 1:63,360; Beland, 1957). Sedimentary rocks are probably more than 7 km thick (Lyons and others, 1980). Comments: Stations set on steel plate in steel culvert (shielded inside with concrete), set on concrete pad on washed bedrock. Confidence: High.
- Chicoutimi-Nord (GSC #16)** 48.490°N 71.012°W
 Precambrian gneiss underlies station site. Paleozoic limestones and shales of unknown thickness crop out approximately 4+ km to west near Lac St. Jean. (scale 1:250,000; Laurin and Sharma, 1975). Comments: Good free-field station. One wall of basement is bedrock. Confidence: High.
- Saint Andre (GSC #17)** 48.325°N 71.992°W
 Precambrian granite underlies station site (scale 1:63,360; Benoit and Valiquette, 1971; scale 1:250,000; Laurin and Sharma, 1975). Comments: Station located out in pasture on 60 meters exposure of bedrock. No noncrystalline rocks in area. Region of low topography. Confidence: High.
- Les Eboulements (GSC #20)** 47.550°N 70.327°W
 Precambrian gneissic granodiorite underlies station site. Anorthosite crops out a few km to northwest (scale 1:63,360; Rondot, 1969; scale 1:250,000; Laurin and Sharma, 1975; Lyons and others, 1980). Comments: Large bedrock exposure. Confidence: High.
- Hebertville, Quebec (HERB)** 48.407°N 71.807°W
 Ordovician Trenton Group Limestone mapped at station site. Limestone occurs as a thin wedge overlying Precambrian granite, and possibly some pyroxene rocks. Granites and pyroxene rocks crop out at surface approximately 2–3 km from station (scale 1:63,360; Benoit and Valiquette, 1971). Borings located 15 km west of the station show soil/glacial till/weathered rock to a depth of approximately 25 meters or less, overlying Trenton Group limestone (30 to 60 meters thick). The limestone is underlain by Precambrian granites/pyroxene rocks to depths of several kilometers. Instrument is buried 0.5 meters deep in soil. Busby (1990) indicates site is underlain by till. Confidence: High.

The following GSC stations were not evaluated.

Edmundston (GSC #12) 46.461°N 68.241°W
Comments: Stations set on steel plate in steel culvert (shielded inside with concrete), set on concrete pad on washed bedrock.

St. Eleuthere (GSC #13) 47.495°N 69.363°W
Comments: Stations set on steel plate in steel culvert (shielded inside with concrete), set on concrete pad on washed bedrock.

St. Georges (GSC #15) 46.140°N 70.580°W
Comments: Stations set on steel plate in steel culvert (shielded inside with concrete), set on concrete pad on washed bedrock.

Rimouski (GSC #18) 48.445°N 68.482°W
Comments: Large bedrock exposure.

Miramichi, New Brunswick (GSC #19) 46.973°N 66.529°W
Comments: Station Loggie Lodge II. Large bedrock exposure. Near station HL.

Ontario and Quebec: Eastern Canadian Telemetered Network

ECTN stations are described as rock sites, with the exception of WEO; thus the thickness extent of unconsolidated deposits at each station is probably less than a few meters. The thickness of possible surficial deposits described for each station reflects the maximum or average thickness observed in the vicinity of each station.

Maniwaki (MIQ-ECTN) 46.37°N 75.97°W
Proterozoic sillimanite-garnet gneiss and quartzite mapped in Maniwaki region. Marbles crop out 0.4 km southeast and 1.5 km northeast of the station. Veneer of Lake Champlain sediments (clay, silt, and sand) is present across most of region but not present at station site (?) (scale 1:63,360; Wilson, 1924). Region of low relief on Canadian Shield. Confidence: High.

LDQ (LDQ-ECTN) 53.81°N 77.43°W
Precambrian granitic rock underlies region. Site is washed bedrock, with less than 1 meter of glacial till. (scale 1:100,000, Vincent, 1985; scale 1:500,000, Shaw, 1942; scale 1:1,500,000, Avramtchev, 1985). Region of low relief east of James Bay on Canadian Shield. Confidence: Moderate to High.

Ottawa (OT-ECTN) 45.39°N 75.72°W
Paleozoic limestone, sandstone, and minor shale (Cambrian-Silurian Black River?) mapped at station site. Surficial geologic map shows erosion terrace on Lake Champlain sediments (clay and silt), overlying glacial till at station site (thickness less than 5 meters?) (scale 1:63,360, Ells, 1901; scale 1:50,000, Richard, 1982a). Region of very low relief in Ottawa. Confidence: High.

Montreal - Brebeuf College (MNT-ECTN) 45.50°N 73.62°W
Montreal region is underlain by Lower Cretaceous Monteregian intrusive rocks and early Paleozoic metasedimentary rocks (sandstone, siltstone, and limestone). Surficial deposits (?) at the station may include about 3 to 6 meters of Champlain Sea sediments (sand and gravel) (scale 1:50,000, Prestand Hode-Keyser, 1977). Region of low relief along St. Lawrence River. Confidence: Moderate to High.

- Manicouagan (MNQ-ECTN)** 50.53°N 68.77°W
Precambrian gneiss/granite mapped throughout the region; presence/extent of glacial sediments at the station is uncertain (scale 1:250,000, Franconi and others, 1975). Region with little topography on Canadian shield. Confidence: Moderate to High.
- GNT (GNT-ECTN)** 46.36°N 72.37°W
Ordovician Pontgrave River Formation (shale and limestone) mapped throughout region; thickness of metasedimentary rocks is more than several km? Pleistocene surficial deposits (including glacial till and overlying Lake Champlain clays and sands) are as much as 60 meters thick in the region; Outcrops of Ordovician rock are reported from the vicinity of the station, and the presence/extent of Pleistocene sediments is uncertain (scale 1:63,360, Gadd, 1959). Broad flat (?) region along southeast side of St. Lawrence River. Confidence: Moderate to High.
- FHO (FHO-ECTN)** 45.46°N 76.22°W
Lower Ordovician March and Oxford formations (dolomite, minor sandstone) mapped at station site west of Ottawa. Proterozoic marble, gneiss, and amphibolite crop out 0.6 km to the south, and middle Ordovician shale, sandstone, and limestone crop out 0.5 km to the northeast of the station (scale 1:50,000, Wilson and others, 1974). Surficial deposits include Lake Champlain sediments (sand, gravel), forming an erosional terrace, thickness uncertain (probably several tens of meters or less) (1:50,000, Richard, 1984). Region of very low relief along a river. Confidence: High.
- GAC (GAC-ECTN)** 45.70°N 75.48°W
Proterozoic paragneiss mapped throughout Ottawa region (scale 1:1,500,000, Avramtchev, 1985). Probable surficial deposits include Lake Champlain sediments (sand, gravel, clay) overlying till; thickness less than ten meters (?) (scale 1:1,000,000, Barnet and others, 1991). Region of low relief on Canadian Shield (?). Confidence: Moderate.
- LPQ (LPQ-ECTN)** 47.34°N 70.01°W
Ordovician Riviere Ouelle Formation (shales and siltstones) underlies station site. Underlain by Kamouraska Formation (quartzite) and Saint Damase Formation (arkose). Rocks are tightly folded into NE-SW trending anticlines and synclines (scale 1:63,360, Hubert, 1973). Thickness of metasedimentary rocks in region is modeled as 7 km (Lyons and others, 1980). Broad flat region along southeast side of St. Lawrence River (elevation 415 ±10 ft). Confidence: High.
- SBQ (SBQ-ECTN)** 45.38°N 71.93°W
Lower Ordovician D'Ascot Formation (dark gray phyllite) mapped at station site. Middle Ordovician De Magog Group (sandstone and slate) crops out 2.2 km northwest of the station. Lower Ordovician/Cambrian D'Ascot Formation (schist and metarhyolite) and Ordovician rhyolite crops out 0.5 km southeast of the station (scale 1:50,000, St. Julian, 1970; scale 1:126,720, Cooke, 1950). Thickness of glacial till not noted, but probably several tens of meters or less. Region of low relief, site on side of low ridge. Confidence: High.
- VDQ (VDQ-ECTN)** 48.23°N 77.97°W
Archean Martic Group (andesite) underlies site. Archean graywache (including tuff, breccia/biotite schist) crops out 1.7 km to the northeast, and Archean biotite-hornblende granodiorite (intrusive relationship) crops out approximately 3 km to the northeast of the station (scale 1:24,000, Norman, 1941; scale 1:36,360, Dawson, 1966). Presence/extent of glacial till uncertain. Broad flat region of low relief on Canadian Shield. Confidence: High.
- WBO (WBO-ECTN)** 45.00°N 75.28°W
Paleozoic limestone, sandstone, and minor shale crop out at surface in vicinity of station; underlain at unknown depth by Precambrian crystalline rock. Surficial geologic map shows a thin veneer (less than 5 meters) of glacial till and Champlain Sea sediments (marine clays and silts) overlying Paleozoic rocks (scale 1:50,000, Richard, 1982b,c). Region of low relief along northwest bank of St. Lawrence River. Confidence: High.

- CKO (CKO-ECTN)** 45.99°N 77.45°W
Proterozoic hornblende-biotite gneiss mapped throughout region (scale 1:1,000,000, Baer and others, 1977). Surficial deposits include glacio-fluvial outwash (sand and gravel) overlying till, thickness of less than 10 meters (?) (scale 1:1,000,000, Barnett and others, 1991). Region of moderately low relief along Ottawa River. Confidence: Moderate.
- TRQ (TRQ-ECTN)** 46.22°N 74.56°W
Proterozoic gneiss mapped at site. Morin Complex (monzanite) crops out 3.2 km to the west, and marbles crop out 3.2 km south of the station (scale 1:125,000, Emslie and Ermanovics, 1975). Station located on the side of Mount Tremblant. Confidence: High.
- GRQ (GRQ-ECTN)** 46.61°N 75.86°W
Proterozoic biotite gneiss mapped throughout region; marbles crop out approximately 1.5 km south of the station (scale 1:253,440, Wynne-Edwards and others, 1966). Area of low relief on Canadian Shield. Presence/extent of glacial sediments uncertain. Confidence: Moderate.
- JAQ (JAQ-ECTN)** 53.80°N 75.72°W
Precambrian igneous and metasedimentary rock underlie station site. Site is washed bedrock, with less than 1 m of glacial till. (scale 1:100,000, Vincent, 1985; scale 1:1,500,000, Avramtchev, 1985). Region of low relief east of James Bay on Canadian Shield. Confidence: Low to Moderate.
- GSQ (GSQ-ECTN)** 48.91°N 67.11°W
Lower Ordovician slates, sandstone, limestone, and quartzite mapped throughout region (scale 1:253,440, McGerrigle, 1953). Rocks are tightly folded into NE-SW trending anticlines and synclines (?). Thickness of metasedimentary rocks is more than several km? Presence/extent of glacial till/sediments at station site uncertain. Broad flat region along southeast side of St. Lawrence Seaway (Gaspé Peninsula). Confidence: Moderate.
- EBN (EBN-ECTN)** 47.54°N 68.24°W
Devonian metasedimentary rocks (siltstone, sandstone, shale) mapped throughout Edmundston region (northwest New Brunswick) (scale 1:638,000, Ferguson and Fyffe, 1985). Presence/extent of glacial/fluvial sediments unknown. Region of low relief (?). Confidence: Moderate.
- GGN (GGN-ECTN)** 45.12°N 66.82°W
Silurian-Devonian rhyolitic-basaltic subaqueous volcanic rocks mapped at station site. Devonian granitic rock crops out approximately 5 km north of the station (scale 1:638,000, Ferguson and Fyffe, 1985). Presence/extent of surficial deposits (soil) unknown. Region of low relief in southwest New Brunswick (north of Bay of Fundy). Confidence: Moderate.
- LMN (LMN-ECTN)** 45.85°N 64.81°W
PreCarboniferous felsic volcanic rocks with basic dikes mapped at station site. Carboniferous shale, sandstone, and conglomerate crop out 0.8 km north of the station (Wright, 1922, 1:62,500; Ferguson and Fyffe, 1985). Presence/extent of surficial deposits (soil) unknown. Region of moderate relief in southeast New Brunswick (north of Bay of Fundy). Confidence: Moderate.
- KLN (KLN-ECTN)** 46.84°N 66.37°W
Cambrian-Ordovician phyllite, slate, quartzite, and schist mapped at station site. Silurian slate, siltstone, graywacke, conglomerate, and andesite crop out about 3 km south of the station (scale 1:50,000, Anderson, 1969). Presence/extent of surficial deposits uncertain. Region of low to moderate relief (elevation 380 ±15 meters). Confidence: High.

- HTQ (HTQ-ECTN)** 49.19°N 68.39°W
Precambrian gneiss mapped throughout the region; a thin veneer (less than 10 meters?) of glacial till/outwash (gravel, sand, and clay) is mapped in the site vicinity, but the presence/absence of sediments at the station site is unknown (scale 1:126,720, Heywood, 1963). Low-lying region along Manicouagan River. Confidence: High.
- WEO (WEO-ECTN)** 44.02°N 78.37°W
Surficial deposits include Pleistocene/Recent glacio-lacustrine sands and gravels, and glacial till with a maximum thickness of 15 meters or less at the station site (Lake Iroquois deposits overlie glacial till; scale 1:63,360, Gravenor, 1953; scale 1:126,720, Gravenor, 1957). Middle Ordovician Cobourg Limestone (Lindsey Formation limestone, thickness of about 200 meters or more) mapped at station site. Limestone is underlain by Precambrian crystalline rock (scale 1:63,360, Liberty, 1960). Region of very low relief on southern Canadian Shield. Confidence: High.
- SUO (SUO-ECTN)** 46.40°N 81.01°W
Hough Lake Group (Mississagi Formation arkose) mapped at station site. Underlain and intruded by Nipissing Gabbro. Middle Proterozoic granitic rock crops out approximately 3 km southeast of the station (scale 1:50,000, Ontario Geological Survey). Glacial till is present in thin discontinuous sheets (thickness of several meters?) across site vicinity. Station may be located on bedrock (scale 1:1,000,000, Barnet and others, 1991). Region of low relief (?) in central Ontario. Confidence: High.
- EEO (EEO-ECTN)** 46.64°N 79.07°W
Archean granitic-dioritic gneiss mapped throughout region (scale 1:253,440, Barlow, 1899). Glacial till is present in thin discontinuous sheets (thickness of several meters?) across site vicinity. Station may be located on bedrock (scale 1:1,000,000, Barnet and others, 1991). Region of low relief on Canadian Shield. Confidence: Moderate.
- DPQ (DPQ-ECTN)** 46.68°N 72.78°W
Archean granite/gneiss underlie the region (scale 1:253,440, Ells, 1900; scale 1:250,000, Laurin and Sharma, 1975). The presence/extent of glacial sediments at the station is uncertain. Station site lies above highest level of Lake Champlain. Region of low relief on Canadian shield, broad valley below rolling hills, northwest of St. Lawrence River. Confidence: Moderate.
- SZO (SZO-ECTN)** 46.44°N 81.50°W
Archean quartz monzonite, granodiorite, diorite, and granite mapped at station site. Mixed intrusive/extrusive volcanic rocks crop out about 2.5 km to the south, and the early Proterozoic Sudbury Igneous Complex (norite, gabbro, granophyre) crops out 0.8 km to the north of the station (scale 1:50,000, Ontario Geological Survey, 1984). Glacial outwash deposits or till in thin discontinuous sheets (thickness of several meters?) mapped in site vicinity. Station may be located on bedrock (scale 1:1,000,000, Barnet and others, 1991). Region of low relief (?) in central Ontario. Confidence: High.
- SWO (SWO-ECTN)** 46.73°N 81.00°W
Region underlain by early Proterozoic Whitewater Group (Sudbury Igneous Complex). Site is located on Onaping Formation Breccias (pieces of older rocks in recrystallized matrix) or other igneous rocks (with or without pieces of older rocks) (scale 1:50,000, Ontario Geological Survey, 1984). Glacial till is present in thin discontinuous sheets (thickness of several meters?) across site vicinity. Station may be located on bedrock (scale 1:1,000,000, Barnet and others, 1991). Region of low relief (?) in central Ontario. Confidence: High.

- A11 (A11-ECTN)** 47.24°N 70.20°W
Cambrian Saint Roche Formation (siltstone, mudstone, shale, arkose, and limestone) mapped at station site. Ordovician Saint Damase Formation (arkose) crops out approximately 1 km southeast of the station. Rocks are tightly folded into NE-SW trending anticlines and synclines (scale 1:63,360, Hubert, 1973). Thickness of metasedimentary rocks in region is modeled as 7 km (Lyons and others, 1980). Broad flat region along southeast side of St. Lawrence River (elevation 160 ±10 ft). Confidence: High.
- A16 (A16-ECTN)** 47.47°N 70.01°W
Ordovician Riviere Ouelle Formation (shales and siltstones) mapped at station site. Underlain by Kamouraska Formation (quartzite). Rocks are tightly folded into NE-SW trending anticlines and synclines (scale 1:63,360, Hubert, 1973). Thickness of metasedimentary rocks in region is modeled as 7 km (Lyons and others, 1980). Presence/extent of glacial/fluvial sediments uncertain. Low-lying region along southeast side of St. Lawrence River (elevation 40 ±10 ft). Confidence: High.
- A21 (A21-ECTN)** 47.70°N 69.69°W
Cambrian-Ordovician metasedimentary rocks (red and green slate and sandstone) mapped throughout region (scale 1:500,000, Dresser, 1911). Rocks at station are probably Ordovician Riviere Ouelle Formation (shales and siltstones) that are underlain by Kamouraska Formation (quartzite), based on detailed mapping to southwest (scale 1:63,360, Hubert, 1973). Rocks are tightly folded into NE-SW trending anticlines and synclines. Thickness of metasedimentary rocks is modeled as 7 km (Lyons and others, 1980). Presence/extent of glacial/fluvial sediments uncertain. Area of low relief southwest of St. Lawrence River. Confidence: Moderate.
- A61 (A61-ECTN)** 47.69°N 70.09°W
Precambrian granitic rocks underlie the La Malbaie region. Station located near onlap contact of St. Lawrence Lowlands sedimentary rocks (scale 1:250,000, Laurin and Sharma, 1975, Lyons and others, 1980). Presence/extent of glacial/fluvial sediments uncertain. Region of low/moderate relief north of St. Lawrence River. Confidence: Low
- A64 (A64-ECTN)** 47.83°N 69.89°W
Precambrian granitic rock mapped throughout region (scale 1:63,360, Miller, 1973). Presence/absence of thin veneer (less than 10 meters) of glacial till is unknown. Region of low relief (?) along northwest bank of St. Lawrence River (elevation 137 meters). Confidence: High.
- A54 (A54-ECTN)** 47.46°N 70.41°W
Precambrian gabbro-anorthosite and charnockite mapped throughout region (scale 1:63,360, Rondot, 1969). Thin sheets (2 meters thick) of Quaternary glacial/fluvial sediments mapped at site, station located on rock outcrop? Region of low/moderate relief about 1.5 km north of St. Lawrence River. Confidence: High.

New Brunswick

General notes on geology of Miramichi region. Throughout most of the region, bedrock is overlain by a thin mantle of Pleistocene glacial till (thickness generally less than 6 meters). Cambrian-Ordovician sedimentary rocks are tightly folded and metamorphosed (greenschist and amphibolite grade). Devonian granitic and other plutonic rocks are massive and relatively undeformed, in contrast to older cataclastic Ordovician granitic/plutonic rocks. There may be a slight difference in velocities between these plutonic rocks. The extent of weathering of bedrock probably is insignificant because of extensive glacial erosion during the Pleistocene. Bedrock is fractured, contains sills and a few mylonitic zones (faults). Pre-Devonian rocks typically are characterized by more extensive deformation (fracturing and intrusions) than younger rocks.

Holmes Lake, New Brunswick (HL) (USGS C9A) 46.945°N 66.594°W

Cambrian/Lower Ordovician quartzite/phyllite underlies station site. Station located near contact with schist/paragneiss. Devonian biotite quartz monzonite (granitic) pluton located 1.5 km north of station (scale 1:50,000; Skinner, 1974). Station may be underlain by pluton at an unknown depth. Station set on massive concrete fireplace hearth (Munro and Pomeroy, 1983). Thickness of glacial till is approximately 5 meters (Mueller and Cranswick, 1985; Bernreuter and others, 1987). Confidence: High.

Mitchell Lake Road, New Brunswick (MR) (USGS CBA) 47.034°N 66.612°W

Devonian granite underlies station to a depth of several kilometers (?). It is a massive granitic pluton that was emplaced into Cambrian/Ordovician sandstone and mudstone (quartzite and phyllite) (scale 1:50,000; Fyffe, 1983; Skinner, 1974). (Maps do not cover station site). Station set on granitic outcrop (Munro and Pomeroy, 1983; Bernreuter and others, 1987). Confidence: Moderate to High.

Loggie Lodge I, New Brunswick (LL) (USGS C7A) 46.969°N 66.529°W

Devonian granite underlies station to a depth of several kilometers (?). It is a massive granitic pluton that was emplaced into Cambrian/Ordovician sandstone and mudstone (quartzite and phyllite) (scale 1:50,000; Fyffe, 1983; Skinner, 1974). Station set on major granitic boulder in glacial till (Munro and Pomeroy, 1983). Till is approximately 5 meters thick, overlying granitic plutonic rocks (Mueller and Cranswick, 1985; Bernreuter and others, 1987). Area of very low relief along Miramichi River. Confidence: High.

Prince Camp, New Brunswick (USGS C8A) 47.008°N 66.501°W

Devonian granite underlies station to a depth of several kilometers (?). It is a massive granitic pluton that was emplaced into Cambrian/Ordovician sandstone and mudstone (quartzite and phyllite) (scale 1:50,000; Fyffe, 1983; Skinner, 1974). Borings at the USGS temporary station show surficial deposits that include 7 meters of till overlying 5 meters of sediment (with faster velocity) (Mueller and Cranswick, 1985). Area of low relief. Confidence: High.

Long Lake, New Brunswick 47.057°N 66.933°W

Ordovician granite (?), intruded into Cambrian/Lower Ordovician quartzite-phyllite-schist-paragneiss, mapped near station site. Station may be underlain primarily by granitic rocks, or by the metasedimentary rocks (scale 1:50,000; Skinner, 1974). (Map does not cover station site). Confidence: Low to Moderate.

Indian Brook I, New Brunswick (IB)

46.978°N 66.581°W

Devonian granite underlies station to a depth of several kilometers (?). It is a massive granitic pluton that was emplaced into Cambrian/Ordovician sandstone and mudstone (quartzite and phyllite). Foliated granite, which was intruded into the sandstone/mudstone during the Ordovician, crops out approximately 1.5 km south of the station (scale 1:50,000; Fyffe, 1983; Skinner, 1974). Station set on granitic boulder in gravel (till) (Munro and Pomeroy, 1983). Thickness of till is approximately 5 meters (Bernreuter and others, 1987). Region of generally low relief. Confidence: High.

Bear Lakes I, New Brunswick (BL)

46.928°N 66.484°W

Cambrian/Lower Ordovician quartzite-phyllite-slate-schist mapped at station site. Devonian biotite quartz monzonite (granite) pluton is exposed 2.5 km northwest of station (scale 1:50,000; Skinner, 1974; Anderson, 1969). Station set on concrete pad on gravel (Munro and Pomeroy, 1983). Gravel/glacial till may be about 5 meters (?) thick. Region of generally low relief. Confidence: High.

Hickey Lake, New Brunswick (HCL) (near GSC 19)

47.006°N 66.546°W

Devonian granite underlies station to a depth of several kilometers (?). It is a massive granitic pluton that was emplaced into Cambrian/Ordovician sandstone and mudstone (quartzite and phyllite) (scale 1:50,000; Fyffe, 1983). Station set on granitic bedrock (Munro and Pomeroy, 1983). Region of generally low relief. Confidence: High.

Indian Brook II, New Brunswick (IB2)

46.993°N 66.596°W

Devonian granite underlies station to a depth of several kilometers (?). It is a massive granitic pluton that was emplaced into Cambrian/Ordovician sandstone and mudstone (quartzite and phyllite). The sandstone/mudstone also contains foliated granite intruded during the Ordovician, and crops out approximately 2 km west of the station (scale 1:50,000; Fyffe, 1983; Skinner, 1974). Station set on granitic bedrock (Munro and Pomeroy, 1983). Region of generally low relief. Confidence: High.

APPENDIX 2.B

EXAMINATION OF VARIANCE OF SEISMOGRAPHIC NETWORK DATA

This appendix presents an analysis of the variability of peak amplitudes of seismographic network data. The empirical estimates of the variability in ENA ground motion amplitudes are derived from a data set that contains a large portion of seismographic network recordings obtained at regional distances, rather than strong motion recordings at distances usually of interest in evaluating seismic hazards from moderate size events. Therefore, it is important to examine the potential for significant differences between the variability in regional seismographic network peak amplitude data and strong motion peak amplitude data.

To examine this issue, statistical analyses were performed utilizing seismographic network data from California which spans the distance range from a few kilometers to several hundred kilometers. Two data sets were used: recent Wood-Anderson amplitudes obtained from the Seismographic Network of the University of California, Berkeley (UCB), and data compiled by Bakun and Joyner (1984) for central California. The variability computed from these data sets were then compared with that obtained for WNA strong motion data and with ENA regional network data.

2.B.1 Seismographic Network Data

U.C. Berkeley Data Set—The U.C. Berkeley Seismographic Station of U.C. Berkeley operates four Wood-Anderson (W-A) instruments at stations ARC (Arcata), MIN (Mineral), BKS (Berkeley), and MHC (Mt. Hamilton). Wood-Anderson amplitude data were selected from the U.C. Berkeley data base from the time period 1984 to 1991 for events that occurred inside a polygon whose apexes are at coordinates (38.5° N, 123.0° W), (35.0° N, 123.0° W), (35.0° N, 120.0° W), (36.5° N, 120.0° W), and (38.5° N, 121.7° W). This zone represents the region of north-central California for which a uniform crustal model is used by UCB for earthquake location. Data from a total of 141 earthquakes were selected for which at least six W-A amplitudes (each horizontal component is counted as one measurement) were measured for each earthquake. The scattergram of Richter magnitude (M_L) and hypocentral distance (R) for this data set is shown on Figure 2.B-1. A plot of the geometrical mean of the two recorded zero-to-peak W-A amplitudes versus hypocentral distance is shown on Figure 2.B-2.

Bakun and Joyner —The Bakun and Joyner (1984) data set consists primarily of synthetic W-A amplitudes calculated from horizontal-component seismograms recorded by the U.S. Geological Survey's (USGS) CALNET (central California seismic network). These amplitude data were supplemented with true W-A amplitudes measured at stations MHC, BKS and MIN operated by U.C. Berkeley. In all, 20 seismograph locations and 106 earthquakes that occurred before 1982 are included in this data set. The scattergram of M_L and R for the Bakun and Joyner data set is also shown on Figure 2.B-1 and a plot of the geometrical mean of the two measured W-A amplitudes versus R is shown on Figure 2.B-2. More detailed descriptions of these earthquakes and the recording stations can be found in Bakun and Joyner (1984).

2.B.2 Data Analysis

The two data sets were analyzed using the random effects regression model described in Section 2.2. The regression relation fit to the data was

$$\log_{10} A_{ij} = c_1 M_i - c_2 R_{ij} + c_3 \log_{10} R_{ij} + \sum_{l=1}^L \delta_{ij} S_l + \eta_i + \varepsilon_{ij} \quad (2.B-1)$$

where A_{ij} (in units of mm) is the geometric mean of the two horizontal W-A amplitudes measured at seismographic station j for earthquake i , R_{ij} is the hypocentral distance, M_i is the Richter magnitude of earthquake i , δ_{ij} is the Kronecker delta, S_l is the station correction term, and L is the total number of station used in regression analysis. The η_i represents inter-event variations from the mean model, and the ε_{ij} represent intra-event variations. The η_i and ε_{ij} are assumed to be independent, normally distributed random variables with variances τ^2 and σ^2 , respectively. In Equation 2.B-1, the first four terms represent the fixed effects and the fifth term represents the random effects.

The algorithm described in Abrahamson and Youngs (1992) was used to estimate the unknown parameters of the mean model, C_k ($k=1,3$), the station correction terms, S_l ($l=1,L$), and the variance terms τ^2 and σ^2 . Parameter estimates and their estimated standard errors are listed in Tables 2.B-1 and 2.B-2 for the UCB and Bakun and Joyner (1984) data sets, respectively. Note that only the relative values of station correction S_l are meaningful. Because Richter magnitudes for these events were determined by the measured W-A amplitudes, the estimates of τ are near zero.

Plots of residuals, defined as ($\log_{10}$ observed – $\log_{10}$ predicted) versus epicentral distance, R , are shown on Figure 2.B-3. Mean residuals and their 95% confidence intervals were then estimated over 10-km distance intervals and the results are plotted versus distance on Figure 2.B-4. When less than 2 data points exist in a distance bin, a mean residual was not computed. Vertical bars in the figures indicate the 95% confidence intervals of the estimated mean residuals. The computed mean residuals shown for the Bakun and Joyner (1984) data set are similar to the results shown on Figure 4 of their paper, indicating that the use of a different algorithm and the geometrical mean of the W-A data yielded very similar results. The positive bias in the mean residuals at distances near 100 km is comparable to the one observed by Bakun and Joyner (1984). This bias is even more prominent in the UCB data set, and may be due to the effects of the velocity gradient in the lower crust (e.g. Burger and others, 1987).

Figure 2.B-5 shows standard deviations of the residuals also computed for 10-km distance intervals. For comparison, standard deviations estimated by Boore (personal communication, 1992) from the Southern California W-A amplitude data set of Hutton and Boore (1987) and Atkinson (personal communication, 1992) from the ECTN data set of Atkinson and Mereu (1992) are also plotted on Figure 2.B-5. The ECTN data are heavily smoothed Fourier amplitudes at 1.0 Hz.

The results shown on Figure 2.B-5 indicate that the standard errors computed for the ECTN data are comparable to those obtained for the California data sets. The two central California data sets and the ECTN data set results show no significant trend in the standard error with distance, while the results for the southern California data set indicate a decrease in standard error with increasing distance. These results indicate that the variability in ground motion amplitude does not increase significantly at regional distances, and that the level of variability in California and southeastern Canada is similar.

The standard errors shown on Figure 2.B-5 were computed with station correction terms applied for individual recording stations, a procedure that is not typically applied to estimation of variability in strong ground motion data. If the individual station correction terms are assumed to represent site-to-site random effects, then the standard deviation of the station corrections can be considered an estimate of the site-to-site variability that is present in strong motion data sets. Assuming independence between the site random effects and the other components of variance, then the combined standard error for the two central California data sets are approximately 0.27 for $\log_{10}$ (amplitudes), or 0.6 for natural log

amplitudes. This value is comparable to the standard error obtained for peak acceleration data from small magnitude California earthquakes by Youngs and others (1990).

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Table 2.B-1

Results of Regression Analysis Using UCB W-A Data

Parameter	Estimate	Estimated Standard Error
C_1	0.9456	0.0209
C_2	1.4552	0.0620
C_3	-0.0012	0.0003
S_1 (BKS)	0.2374	0.1194
S_2 (MHC)	-0.0099	0.1147
S_3 (MIN)	0.1863	0.1135
S_4 (ARC)	-0.0861	0.1255
τ	~0.0000	-
σ	0.2211	0.0090

Table 2.B-2

Results of Regression Analysis Using Bakun and Joyner's (1984) Data

Parameter	Estimate	Estimated Standard Error
C ₁	0.9591	0.0153
C ₂	1.1910	0.1217
C ₃	-0.0027	0.0008
S ₁	-0.1098	0.1735
S ₂	-0.5007	0.1627
S ₃	-0.2280	0.1727
S ₄	-0.1536	0.1528
S ₅	-0.3088	0.1692
S ₆	-0.3740	0.1676
S ₇	-0.2181	0.1347
S ₈	0.1594	0.1471
S ₉	-0.4286	0.1611
S ₁₀	-0.3719	0.1608
S ₁₁	-0.1007	0.1663
S ₁₂	-0.2600	0.1751
S ₁₃	-0.1730	0.1767
S ₁₄	-0.3713	0.1358
S ₁₅	0.2361	0.1461
S ₁₆	0.0022	0.1805
S ₁₇	-0.1526	0.1545
S ₁₈ (BKS)	-0.2384	0.1690
S ₁₉ (MHC)	-0.5563	0.1702
S ₂₀ (MIN)	-0.0725	0.1622
τ	~0.0000	-
σ	0.1977	0.0065

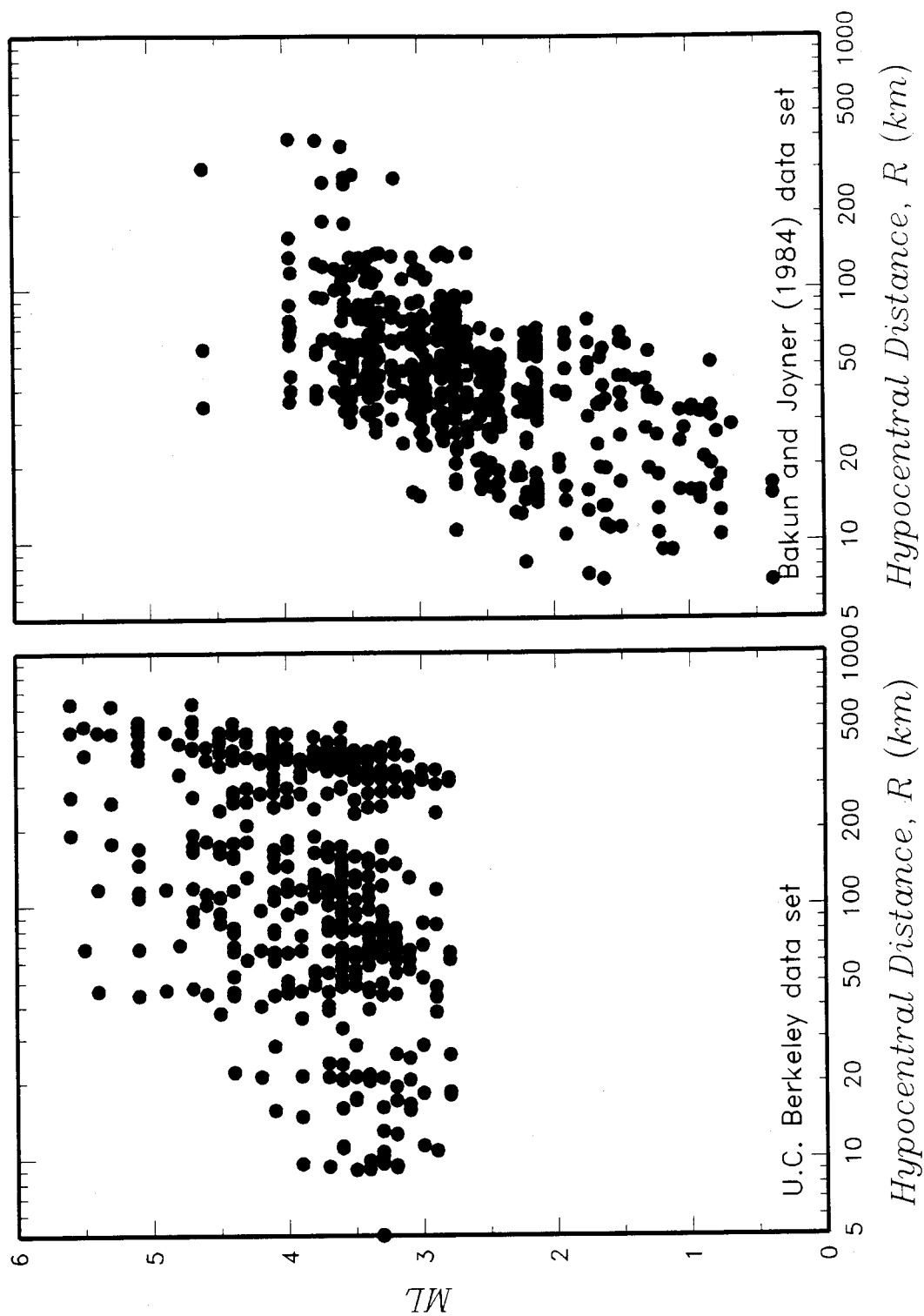


Figure 2.B-1
Magnitude-distance scattergrams of the U.C. Berkeley and Bakun and Joyner (1984) CALNET data analyzed in this study.

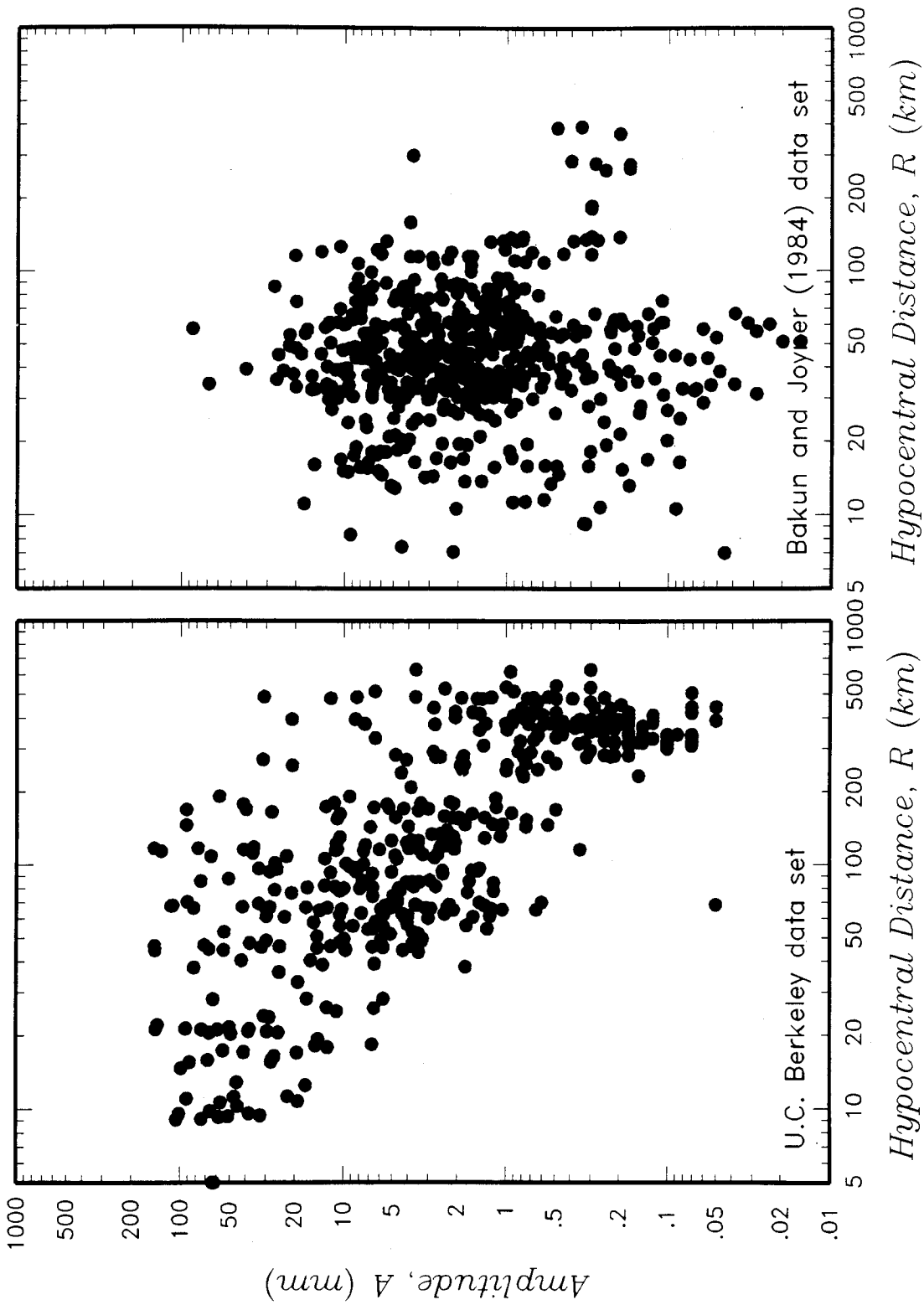


Figure 2.B-2
Distance-amplitude scattergrams of the U.C. Berkeley and Bakun and Joyner (1984) data sets analyzed in the study.

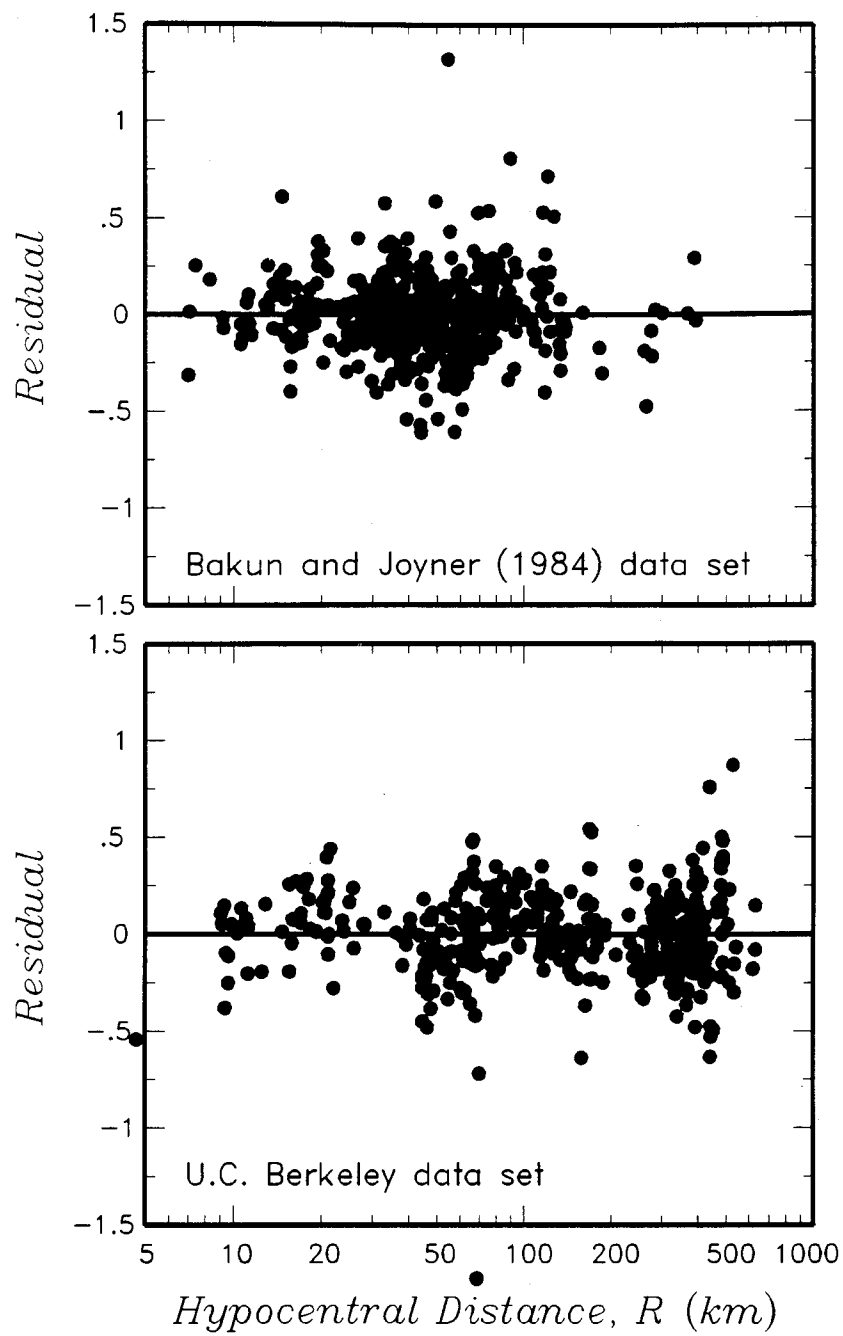


Figure 2.B-3
Residuals from fits of (2.B-1) to UCB and Bakun and Joyner data sets.

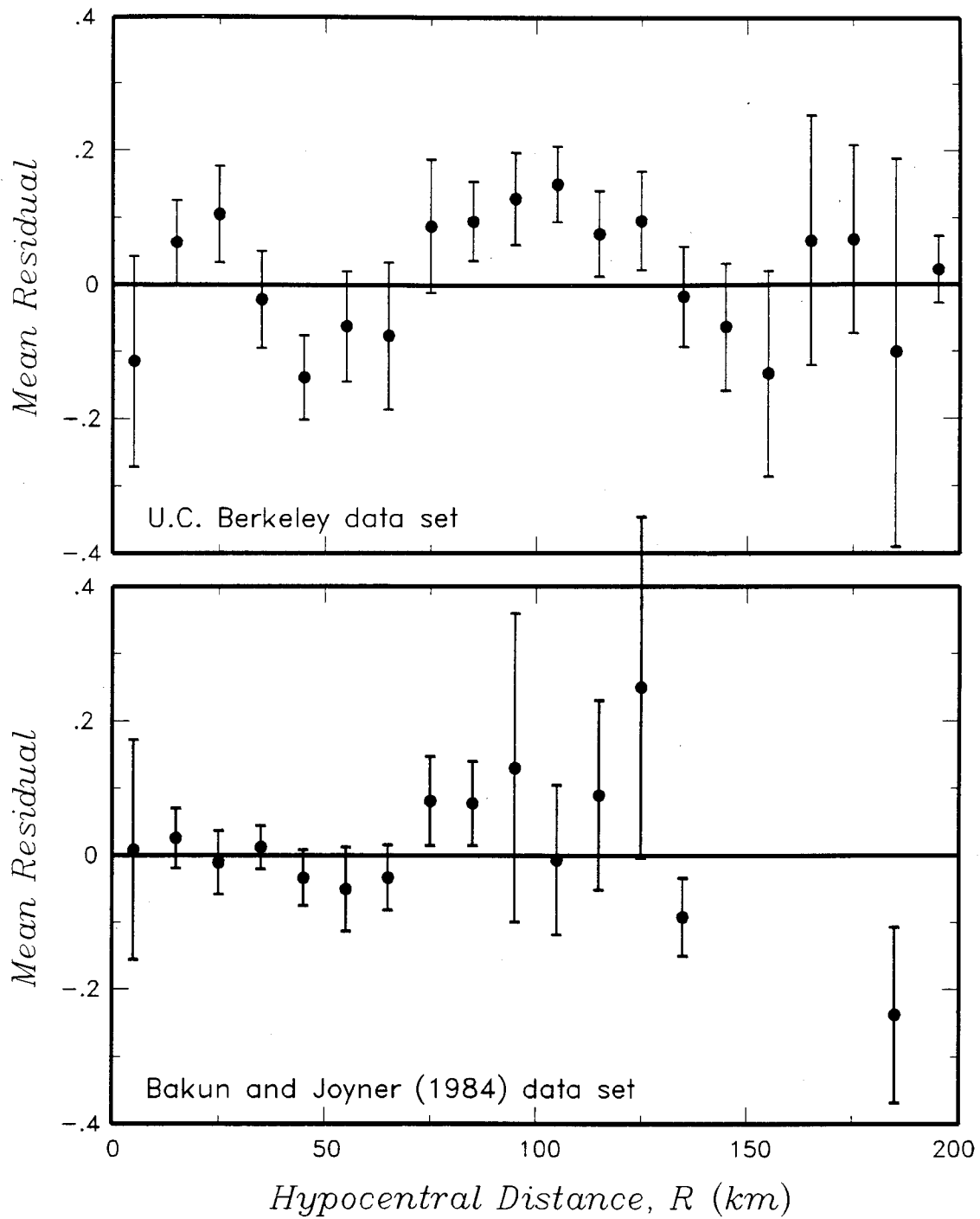


Figure 2.B-4

Mean residuals for 10-km distance intervals from fits of (2.B-1) to UCB and Bakun and Joyner data sets. Error bars show 95% confidence interval.

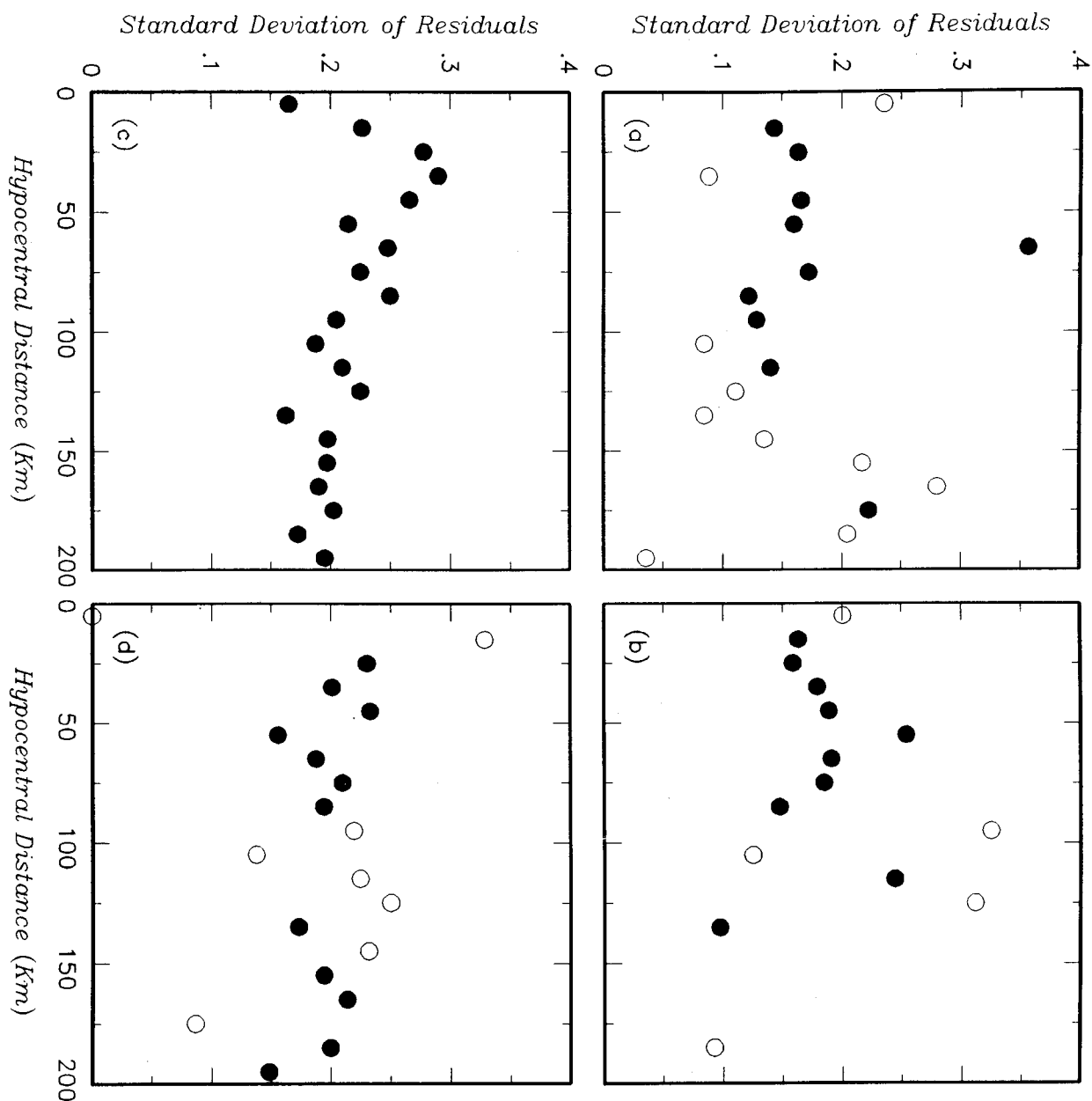


Figure 2.B-5
Standard error of residuals for 10 km distance intervals computed for: (a) U.C. Berkeley data set, (b) Bakun and Joyner (1984) data set, (c) Hutton and Boore (1987) data set, and (d) Atkinson and Mereu (1992) data set. Error estimates for distance intervals having less than 10 data are plotted as open circles.

APPENDIX 2.C

SOIL AMPLIFICATION AND VERTICAL-TO-HORIZONTAL RATIOS FROM ANALYSIS OF STRONG MOTION DATA FROM ACTIVE TECTONIC REGIONS

This appendix presents the results of analyses of strong motion data to evaluate soil/rock amplification ratios and vertical/horizontal ratios for ground motions from moderate/to large earthquakes (M 6 and greater) in active regions.

2.C.1 Strong Motion Data Base

Table 2.C-1 lists the strong motion data analyzed in this study. The data represent the available digitized strong motion recordings from earthquakes of magnitude M 6 and greater recorded on free field conditions (in instrument shelters or on the ground floor of small buildings of light construction without basements). The data were divided into three generalized site conditions: rock, shallow soil (less than 20 m to rock) and deep soil (>20 m to rock). Figure 2.C-1 shows the distribution of the data set in terms of magnitude, distance, and site classification. The records were specially processed to examine the ground motions at high and low frequencies (Silva and Abrahamson, in preparation).

2.C.2 Site Amplification

The effects of shallow and deep soils on strong ground motion were evaluated by fitting an attenuation model to the peak acceleration and 5%-damped response spectral ordinates of the form:

$$\ln(y_{ij}) = C_1 + C_2 m_i + C_3 \ln(r_{ij} + e^{(C_4 + C_5 m_i)}) + Z_{SS} A^{SS} + Z_{DS} A^{DS} + \eta_i + \varepsilon_{ij} \quad (2.C-1)$$

$$A^{SS} = C_6 + C_7 \ln(y_{ij}^{rock})$$

$$A^{DS} = C_8 + C_9 \ln(y_{ij}^{rock})$$

where y_{ij} is the peak ground motion parameter of the i^{th} earthquake of magnitude m_i recorded at distance r_{ij} , Z_{SS} is 1 for shallow soil sites and 0 otherwise, Z_{DS} is 1 for deep soil sites and 0 otherwise, A^{SS} and A^{DS} are site amplification functions that are dependent upon the level of the equivalent rock site ground motion level and C_1 through C_9 are constants fit to the data. The regression analyses were conducted utilizing a *random effects* regression formulation (Brillinger and Preisler, 1985; Abrahamson and Youngs, 1992) which partitions the total error in Equation 2.C-1 into an inter-event or earthquake-to-earthquake component, η_i , and an intra-event or within earthquake component, ε_{ij} . Analyses were conducted for the entire data set and for a data set with the data from the SMART1 array on Taiwan removed. The SMART1 data represent a mixture of crustal and subduction zone earthquakes and may have different characteristics to that of the rest of the data set.

The formulation for the amplification functions A^{SS} and A^{DS} allows for nonlinear site effects. Figure 2.C-2 shows the results of fitting Equation 2.C-1 to peak acceleration data in terms of the relationship of peak acceleration on soil sites to peak acceleration on rock. The relationships between peak acceleration on soil and peak acceleration on rock have the familiar shape indicating amplification at low rock motion levels and de-amplification at high rock motion levels. Also, the shallow sites show smaller soil amplification effects than do the deep soil sites.

Figures 2.C-3 through 2.C-7 show the results of fits to the data. Each figure shows the predicted response spectral accelerations for a magnitude M 6.5 and distances from 1 to 50 km for the three site conditions together with the soil/rock spectral ratios. The results show increasing soil amplification as the distance from the source increases.

2.C.3 Ratio of Vertical to Horizontal Motion

The data set listed in Table 2.C-1 was used to evaluate the ratio of vertical to horizontal peak accelerations and spectral accelerations. For each record a set of vertical/horizontal amplitude ratios was computed by dividing the vertical spectral ordinate by the geometric mean of the horizontal spectral ordinates. Two types of analyses of this data were then conducted: statistical analysis of subsets of the data corresponding to specified distance and magnitude intervals, and regression analyses of the entire data set. Figure 2.C-8 shows the magnitude-distance scattergram for the data set and the subdivision of the data into three magnitude intervals and four distance intervals. The statistics of the log of the vertical to horizontal spectral ratios were then computed for the data lying within each magnitude-distance bin. The resulting estimates of the mean log spectral ratio and its 95%-confidence interval are shown on Figures 2.C-9a, 2.C-9b, and 2.C-9c by the solid points and error bars.

The second analysis involved a regression analysis of the entire data set using the formulation

$$\ln\left(\frac{y_{ij}^{vertical}}{y_{ij}^{horizontal}}\right) = C_1 + C_2 m_i + C_3 \ln\left(r_{ij} + e^{(C_4 + C_5 m_i)}\right) + C_6 Z_{SS} + C_7 Z_{DS} + \eta_i + \varepsilon_{ij} \quad (2.C-2)$$

The form of Equation 2.C-2 allows for distance, magnitude, and site effects on the ratio of vertical to horizontal motions. Two regression analyses were performed, one including site effects (terms $C_6 Z_{SS}$ and $C_7 Z_{DS}$) and one ignoring site effects.

The fitted regression model was then used to predict the mean log spectral ratio for magnitude-distance pairs corresponding to the central point of the data for each of the twelve magnitude-distance subsets used in the statistical analyses. The resulting spectral ratios are compared to the statistical analyses results on Figures 2.C-9a, 2.C-9b, and 2.C-9c. The results indicate that soil effects have only a little effect on the vertical/horizontal spectral ratios at high frequencies. At low frequencies, in the vertical/horizontal, spectral ratios are consistently lower on deep soils than on rock or shallow soils.

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Table 2.C-1

Strong Motion Data Base used in Analysis of Site Amplification and Vertical/Horizontal Ratios

Earthquake	Date	M	M _{INS}	ST.NO.	Station	SC	ED(km)	CD(km)	Comp	PGA(g)	Comp	PGA(g)	Comp	PGA(g)
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1013	PARKFIELD C 2WA	DS	47.3	000	0.1090	090	0.1137	090	0.1137	0.0400
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1014	PARKFIELD C 5W	SS	51.2	270	0.1397	360	0.1359	360	0.1359	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1015	PARKFIELD C 8W	SS	54.2	000	0.1009	270	0.1003	270	0.1003	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1016	PARKFIELD C 12W	SS	58.2	270	0.0436	360	0.0467	360	0.0467	0.0200
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1018	PARKFIELD C 4AW	DS	50.1	000	0.0512	090	0.0691	090	0.0691	0.0200
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1097	PARKFIELD C 2E	R	45.4	000	0.0259	090	0.0373	090	0.0373	0.0200
Coalinga, CA	1983.05.02	6.5	6.5	USGS 11628	PVPP BASEMENT	DS	6.8	045	0.3000	135	0.2716	135	0.2716	0.2100
Coalinga, CA	1983.05.02	6.5	6.5	USGS 1162S	PVPP SWITCHYARD	DS	6.8	045	0.5956	135	0.5246	135	0.5246	0.3700
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1387	PARKFIELD FZ 12	DS	30.4	090	0.1100	180	0.1121	180	0.1121	0.0700
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1405	PARKFIELD VC 3W	DS	31.7	020	0.0985	110	0.1368	110	0.1368	0.0600
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1592	PARKFIELD FZ 1	DS	45.2	000	0.1406	090	0.1105	090	0.1105	0.0400
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1593	PARKFIELD FZ 3	DS	39.6	000	0.1398	090	0.1637	090	0.1637	0.0500
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1616	PARKFIELD C 3W	DS	48.1	000	0.0977	090	0.0839	090	0.0839	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1617	PARKFIELD C 4W	DS	49.0	000	0.1326	090	0.1306	090	0.1306	0.0400
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1619	PARKFIELD C 6W	DS	52.7	000	0.1313	090	0.0952	090	0.0952	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1620	PARKFIELD C 1E	DS	46.4	000	0.0905	090	0.0889	090	0.0889	0.0600
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1621	PARKFIELD C 3E	R	43.5	000	0.0437	090	0.0460	090	0.0460	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1623	PARKFIELD SC 2E	R	39.8	000	0.0606	090	0.0895	090	0.0895	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1624	PARKFIELD SC 3E	DS	36.4	000	0.1508	090	0.1057	090	0.1057	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1625	PARKFIELD SC 4E	R	33.9	000	0.0721	090	0.0658	090	0.0658	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1626	PARKFIELD FZ 2	DS	41.3	000	0.1162	090	0.1331	090	0.1331	0.0400
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1627	PARKFIELD FZ 4	SS	36.3	000	0.0668	090	0.1197	090	0.1197	0.0500
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1628	PARKFIELD FZ 6	SS	34.4	000	0.0543	090	0.0559	090	0.0559	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1629	PARKFIELD FZ 7	DS	32.7	000	0.1224	090	0.1190	090	0.1190	0.0500
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1630	PARKFIELD FZ 8	SS	31.6	000	0.1322	090	0.1164	090	0.1164	0.0500
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1631	PARKFIELD FZ 9	R	32.8	000	0.0492	090	0.0505	090	0.0505	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1632	PARKFIELD FZ 10	R	31.7	000	0.0727	090	0.1310	090	0.1310	0.0400
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1633	PARKFIELD FZ 11	SS	35.6	000	0.0777	090	0.0869	090	0.0869	0.0400
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1634	PARKFIELD FZ 14	DS	30.1	000	0.2619	090	0.2736	090	0.2736	0.1000
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1635	PARKFIELD FZ 15	SS	29.6	000	0.1916	090	0.1231	090	0.1231	0.0800
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1636	PARKFIELD FZ 16	DS	28.1	000	0.1830	090	0.1430	090	0.1430	0.0600
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1637	PARKFIELD GH 1W	DS	37.1	000	0.1191	090	0.0647	090	0.0647	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1638	PARKFIELD GH 2W	SS	39.0	000	0.0830	090	0.0742	090	0.0742	0.0400
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1639	PARKFIELD GH 3W	R	40.8	000	0.1368	090	0.1219	090	0.1219	0.0700
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1641	PARKFIELD GH 4W	SS	42.8	000	0.0561	090	0.0975	090	0.0975	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1642	PARKFIELD GH 5W	SS	45.2	000	0.0731	090	0.0539	090	0.0539	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1643	PARKFIELD GH 6W	DS	49.3	000	0.0625	090	0.0691	090	0.0691	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1644	PARKFIELD GH 2E	DS	35.2	000	0.0745	090	0.0810	090	0.0810	0.0400
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1645	PARKFIELD GH 3E	DS	32.1	000	0.0939	090	0.0719	090	0.0719	0.0500
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1646	PARKFIELD VC 1W	DS	28.9	000	0.0806	090	0.0868	090	0.0868	0.0700
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1647	PARKFIELD VC 2W	DS	30.1	000	0.0781	090	0.0886	090	0.0886	0.0600
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1648	PARKFIELD VC 4W	SS	34.0	000	0.0561	090	0.0387	090	0.0387	0.0300
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1649	PARKFIELD VC 5W	DS	36.6	090	0.0645	090	0.0480	090	0.0480	0.0400
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1649	PARKFIELD VC 6W	DS	40.5	000	0.0536	090	0.0763	090	0.0763	0.0400

Table 2.C-1 (continued)

Strong Motion Data Base used in Analysis of Site Amplification and Vertical/Horizontal Ratios

Earthquake	Date	M	M _{INS}	ST.NO.	Station	SC	ED(km)	CD(km)	Comp	PGA(g)	Comp	PGA(g)	Comp	PGA(g)
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 1650	PARKFIELD VC 1E	SS		26.2	000	0.1772	090	0.2302	090	0.0800
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 404	SLACK CANYON	R		28.7	045	0.1731	315	0.1361	VRT	0.0500
Coalinga, CA	1983.05.02	6.5	6.5	CDMG 491	CANTUA CREEK SCHOOL	DS		25.9	270	0.2250	360	0.2883	VRT	0.1000
Friuli Seq.	1976.05.06	6.2	6.2	ITAL BAR	BARCIS	SS	54.6		000	0.0289	090	0.0296	VRT	0.1000
Friuli Seq.	1976.05.06	6.2	6.2	ITAL COD	CODROIPO	DS	40.7		000	0.0619	090	0.0905	VRT	0.0300
Friuli Seq.	1976.05.06	6.2	6.2	ITAL CON	CONEGLIANO	DS	85.7		000	0.0491	090	0.0685	VRT	0.0200
Friuli Seq.	1976.05.06	6.2	6.2	ITAL FEL	FELTRE	R	107.6		000	0.0326	090	0.0376	VRT	0.0200
Friuli Seq.	1976.05.06	6.2	6.2	ITAL TOL	TOLMEZZO	SS	23.3		000	0.3513	090	0.3148	VRT	0.2700
Friuli Seq. (A26)	1976.09.15	6.1	6.1	ITAL BUI	BUIA	DS	9.8	11.0	000	0.1098	090	0.0907	VRT	0.0700
Friuli Seq. (A26)	1976.09.15	6.1	6.1	ITAL COD	CODROIPO	DS	39.6	39.9	000	0.0296	090	0.0190	VRT	0.0100
Friuli Seq. (A26)	1976.09.15	6.1	6.1	ITAL FCO	FORGARIA CORNINO	SS	15.9	16.7	000	0.2599	090	0.2122	VRT	0.1000
Friuli Seq. (A26)	1976.09.15	6.1	6.1	ITAL SRO	SAN ROCCO	R	15.9	16.7	000	0.0597	090	0.1344	VRT	0.0600
Gazli, USSR	1976.05.17	6.8	7.3	USSR KAR	KARAKYR	R		3.5	000	0.6249	090	0.7198	VRT	1.3300
Georgia, USSR	1991.06.15		6.2	USSR AMB	AMBRALAURI	R	73.0		000	0.0179	090	0.0163	VRT	0.0100
Georgia, USSR	1991.06.15		6.2	USSR BAZ	BAZA	DS	48.0		000	0.0335	090	0.0383	VRT	0.0200
Georgia, USSR	1991.06.15		6.2	USSR IRI	IRI	DS	35.0		000	0.1180	090	0.1109	VRT	0.0500
Georgia, USSR	1991.06.15		6.2	USSR ONI	ONI	DS	51.0		000	0.0755	090	0.0455	VRT	0.0200
Georgia, USSR	1991.06.15		6.2	USSR ZEM	ZEM	DS	56.0		000	0.0606	090	0.0649	VRT	0.0300
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 286	SUPERSTITION MNT. CAMERA	R	26.0	26.0	050	0.1092	135	0.1954	VRT	0.0800
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 412	EL CENTRO ARRAY 10	DS	8.6	8.6	045	0.1710	320	0.2235	VRT	0.1100
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5028	EL CENTRO ARRAY 7	DS	0.6	0.6	140	0.3375	230	0.4629	VRT	0.5400
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5051	PARACHUTE TEST SITE	DS	14.2	14.2	225	0.1113	315	0.2038	VRT	0.1600
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5052	PLASTER CITY	DS	32.0	32.5	045	0.0424	135	0.0569	VRT	0.0300
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5053	CALEXICO	DS	10.6	10.6	225	0.2749	315	0.2020	VRT	0.1900
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5054	BONDS CORNER	DS	4.0	2.8	140	0.5884	230	0.7747	VRT	0.4200
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5055	HOLTVILLE POST OFF.	DS	7.5	7.5	225	0.2525	315	0.2208	VRT	0.2300
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5056	EL CENTRO ARRAY 1	DS	15.5	15.5	140	0.1395	230	0.1344	VRT	0.0600
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5058	EL CENTRO ARRAY 11	DS	12.6	12.6	140	0.3640	230	0.3796	VRT	0.1400
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5059	EL CENTRO ARRAY 13	DS	21.9	21.9	140	0.1170	230	0.1391	VRT	0.0500
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5060	BRAWLEY AIRPORT	DS	8.5	8.5	225	0.1600	315	0.2198	VRT	0.1500
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5061	CALIPATRIA FIRE ST.	DS	23.8	23.8	225	0.1282	315	0.0785	VRT	0.0600
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5066	COACHELLA CANAL 4	DS	49.3	49.3	045	0.1154	135	0.1280	VRT	0.0400
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5115	EL CENTRO ARRAY 2	DS	10.4	10.4	140	0.3151	230	0.4105	VRT	0.1096
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5157	EL CENTRO ARRAY 3	SS	12.7	10.9	140	0.2663	230	0.2211	VRT	0.1300
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5158	EL CENTRO ARRAY 6	DS	1.3	1.4	140	0.4105	230	0.4391	VRT	1.6600
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5159	EL CENTRO ARRAY 8	DS	3.5	3.5	140	0.6017	230	0.4540	VRT	0.4400
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 5165	DIFFERENTIAL ARRAY	DS	5.3	5.3	230	0.4798	270	0.3521	VRT	0.7100
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 6004	CERRO PRIETO	R	22.0	26.5	147	0.1691	147	0.2118	147	0.1600
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 6605	DELTA	DS	29.0	33.0	262	0.2378	352	0.3511	VRT	0.1400
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 6610	VICTORIA	DS	42.0	30.9	075	0.1217	345	0.1665	VRT	0.0600
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 6616	AEROPUERTO	DS	5.0	5.0	045	0.3265	315	0.2594	VRT	0.1400
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 6617	CUCAPAH	DS	12.7	5.7	085	0.3085			VRT	0.1401
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 6619	SAHOP CASA FL.	DS	9.7	9.8	000	0.2420			VRT	0.3800
Imp. Val., CA	1979.10.15	6.5	6.5	USGS 6621	CHIHUAHUA	DS	17.0	18.4	012	0.2704	282	0.2541	VRT	0.2200

Table 2.C-1 (continued)

Strong Motion Data Base used in Analysis of Site Amplification and Vertical/Horizontal Ratios

Earthquake	Date	M	M _{INS}	ST.NO.	Station	SC	ED(km)	CD(km)	Comp	PGA(g)	Comp	PGA(g)	Comp	PGA(g)	Comp	PGA(g)
Imp.Val., CA	1979.10.15	6.5	6.5	UCSD 6622	COMPUERTAS	DS	23.0	23.7	015	0.1862	285	0.1473	VRT	0.0700	VRT	0.0700
	1979.10.15	6.5	6.5	USGS 931	EL CENTRO ARRAY 12	DS	18.2	18.2	140	0.1433	230	0.1163	VRT	0.0700	VRT	0.0700
	1979.10.15	6.5	6.5	USGS 952	EL CENTRO ARRAY 5	DS	1.0	1.0	140	0.5191	230	0.3786	VRT	0.5400	VRT	0.5400
	1979.10.15	6.5	6.5	USGS 955	EL CENTRO ARRAY 4	DS	4.2	4.2	140	0.4854	230	0.3600	VRT	0.2500	VRT	0.2500
Kern County, CA	1952.07.21	7.4	7.4	USGS 135	HOLLYWOOD STORAGE LOT	DS	119.5	120.5	090	0.0438	180	0.0568	VRT	0.0200	VRT	0.0200
	1992.07.23	7.3		SCE	LUCERNE VALLEY	SS		1.7	HR1	0.7161	HR2	0.8459	VRT	0.7600	VRT	0.7600
Landers, CA	1992.07.23	7.3		SCE	COOLWATER GENERAT. ST.	DS		19.0	HR1	0.2829	HR2	0.4171	VRT	0.1700	VRT	0.1700
	1989.10.17	7.0	7.1	USGS 1001	APEEL 1-FOSTER CITY	DS	63.0	49.5	270	0.1070	360	0.1160	VRT	0.0700	VRT	0.0700
Loma Prieta, Ca	1989.10.17	7.0	7.1	USGS 1002	APEEL 1-REDWOOD CITY	DS	47.9	47.9	043	0.2736	133	0.2201	VRT	0.0800	VRT	0.0800
Loma Prieta, Ca	1989.10.17	7.0	7.1	USGS 1032	HOLLISTER-SAGO VAULT	R	30.6	30.6	270	0.0360	360	0.0595	VRT	0.0400	VRT	0.0400
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1107	APEEL 10-SKYLINE	R	63.0	47.7	000	0.1026	090	0.0885	VRT	0.0400	VRT	0.0400
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1121	APEEL 2E-HAYW.MUIR SCH	DS	57.4	57.4	000	0.1706	090	0.1385	VRT	0.1000	VRT	0.1000
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1160	APEEL 7-PULGAS	R	63.0	47.7	000	0.1565	090	0.0875	VRT	0.0600	VRT	0.0600
Loma Prieta, Ca	1989.10.17	7.0	7.1	USGS 1161	APEEL 9-CRYSTAL SPR.RES.	R	46.9	46.9	137	0.1128	227	0.0485	VRT	0.1000	VRT	0.1000
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1219	APEEL 3E-HAYWARD CSUH	R	71.0	53.0	000	0.0779	090	0.0841	VRT	0.0500	VRT	0.0500
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1250	GILROY-GAVILAN COLL.	SS	29.0	15.0	067	0.3570	337	0.3252	VRT	0.1900	VRT	0.1900
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1251	CORRALITOS	SS	7.0	5.1	000	0.6440	090	0.4795	VRT	0.4600	VRT	0.4600
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1280	POINT BONITA	R	104.0	85.0	207	0.0713	297	0.0725	VRT	0.0300	VRT	0.0300
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1299	FREMONT-MISSION SJ	SS	55.0	39.0	000	0.1244	090	0.1057	VRT	0.0800	VRT	0.0800
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1300	SARATOGA:ALOHA AVE	DS	27.0	13.0	000	0.5125	090	0.3242	VRT	0.3900	VRT	0.3900
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1301	AGNEWS STATE HOSPITAL	DS	40.0	28.2	000	0.1720	090	0.1591	VRT	0.0900	VRT	0.0900
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1342	TREASURE ISLAND	DS	98.0	79.0	000	0.1001	090	0.1592	VRT	0.0200	VRT	0.0200
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1376	CAPITOLA	DS	9.0	14.0	000	0.5077	090	0.4312	VRT	0.5400	VRT	0.5400
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1378	WOODSIDE	SS	55.0	36.0	000	0.0804	090	0.0821	VRT	0.0500	VRT	0.0500
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1379	SF-DIAMOND HEIGHTS	R	73.0	73.0	000	0.0976	090	0.1129	VRT	0.0400	VRT	0.0400
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1380	SF-PACIFIC HEIGHTS	R	97.0	78.0	270	0.0609	360	0.0471	VRT	0.0300	VRT	0.0300
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1384	SANTA CRUZ UCSC LICK OBS	R	16.0	17.9	000	0.4589	090	0.4148	VRT	0.3700	VRT	0.3700
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1397	SF-RINCON HILL	R	95.0	76.0	000	0.0784	090	0.0916	VRT	0.0300	VRT	0.0300
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1401	YERBA BUENA ISLAND	R	95.0	77.0	000	0.0289	090	0.0678	VRT	0.0300	VRT	0.0300
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1408	GILROY #1	R	29.0	11.2	000	0.4108	090	0.4732	VRT	0.2100	VRT	0.2100
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1409	GILROY #2	DS	30.0	12.7	000	0.3673	090	0.3223	VRT	0.2900	VRT	0.2900
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1410	GILROY #3	DS	31.0	14.4	000	0.5551	090	0.3674	VRT	0.3400	VRT	0.3400
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1411	GILROY #4	DS	32.0	16.1	000	0.4165	090	0.2123	VRT	0.1600	VRT	0.1600
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1413	GILROY #6	R	35.0	19.9	000	0.1257	090	0.1703	VRT	0.1000	VRT	0.1000
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1414	SALINAS:OHM & WORK	DS	46.0	32.6	160	0.0912	250	0.1119	VRT	0.1000	VRT	0.1000
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1421	SAGO SOUTH B: SURFACE	R	54.0	39.0	261	0.0726	351	0.0671	VRT	0.0600	VRT	0.0600
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1422	HALLS VALLEY	DS	37.0	31.6	000	0.1326	090	0.1126	VRT	0.0600	VRT	0.0600
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1445	COYOTE LAKE DAM:SW ABUT.	R	31.0	21.8	195	0.1512	285	0.4838	VRT	0.0800	VRT	0.0800
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1454	SF-PRESIDIO	R	98.0	79.0	000	0.0995	090	0.1999	VRT	0.0600	VRT	0.0600
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1455	SF INT. AIRPORT	DS	79.0	60.0	000	0.2361	090	0.3293	VRT	0.0700	VRT	0.0700
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1456	OAKLAND-TITLE&TRUST	DS	92.0	73.0	180	0.1952	270	0.2435	VRT	0.1500	VRT	0.1500
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1467	BELMONT	R	49.9	49.9	000	0.1077	090	0.1103	VRT	0.0400	VRT	0.0400
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1469	PALO ALTO: 1900 EMBARC.	DS		36.1	000	0.2037	090	0.2131	VRT	0.0800	VRT	0.0800
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1499	PIEDMONT JH-GROUND	R	93.0	74.0	045	0.0840	315	0.0711	VRT	0.0300	VRT	0.0300

Table 2.C-1 (continued)

Strong Motion Data Base used in Analysis of Site Amplification and Vertical/Horizontal Ratios

Earthquake	Date	M	M _{INS}	SI.NO.	Station	SC	ED (km)	CD (km)	Comp	PGA (g)	Comp	PGA (g)	Comp	PGA (g)	Comp	PGA (g)
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1595	GILROY #7	SS	40.0	28.0	000	0.2255	090	0.3228	090	0.1200	VRT	0.1200
Loma Prieta, Ca	1989.10.17	7.0	7.1	USGS 1601	PALO ALTO:SLAC LAB	R	36.3	36.3	270	0.1943	360	0.2776	360	0.0900	VRT	0.0900
Loma Prieta, Ca	1989.10.17	7.0	7.1	USGS 1652A	ANDERSON DAM L.ABUTM.	R	21.4	21.4	250	0.0637	340	0.0771	340	0.0500	VRT	0.0500
Loma Prieta, Ca	1989.10.17	7.0	7.1	USGS 1652B	ANDERSON DAM DAMSTRM.	DS	21.4	21.4	270	0.2439	360	0.2398	360	0.1500	VRT	0.1500
Loma Prieta, Ca	1989.10.17	7.0	7.1	USGS 1656	HOLLISTER DIFF. ARRAY	DS	45.0	25.8	165	0.2686	255	0.2789	255	0.1500	VRT	0.1500
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1666	BERKELEY LBL	SS	99.0	80.0	000	0.0487	090	0.1171	090	0.0400	VRT	0.0400
Loma Prieta, Ca	1989.10.17	7.0	7.1	USGS 1678	SF-GOLDEN GATE BRIDGE	R	85.1	85.1	270	0.2334	360	0.1232	360	0.0600	VRT	0.0600
Loma Prieta, Ca	1989.10.17	7.0	7.1	USGS 1686	FREMONT-EMERSON COURT	SS	43.4	43.4	090	0.1916	180	0.1409	180	0.0700	VRT	0.0700
Loma Prieta, Ca	1989.10.17	7.0	7.1	USGS 1695	SUNNYVALE:COLTON AVE	DS	28.8	28.8	270	0.2073	360	0.2086	360	0.1000	VRT	0.1000
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1707	HAYWARD-BART STA	DS	73.0	55.0	220	0.1563	310	0.1595	310	0.0800	VRT	0.0800
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 1716	RICHMOND CITY HALL	DS	108.0	93.1	190	0.1244	280	0.1057	280	0.0300	VRT	0.0300
Loma Prieta, Ca	1989.10.17	7.0	7.1	CDMG 504	COYOTE LAKE DAM:DNST	DS	31.0	22.3	195	0.1604	285	0.1791	285	0.0900	VRT	0.0900
Loma Prieta, Ca	1989.10.17	7.0	7.1	UCSC BRAN	BRAN	R		10.3	000	0.4528	090	0.5011	090	0.5100	VRT	0.5100
Loma Prieta, Ca	1989.10.17	7.0	7.1	UCSC LGPC	LGPC	R		6.1	000	0.5633	090	0.6053	090	0.8900	VRT	0.8900
Loma Prieta, Ca	1989.10.17	7.0	7.1	UCSC UCSC	UCSC	R		18.1	000	0.3091	090	0.3956	090	0.2200	VRT	0.2200
Loma Prieta, Ca	1989.10.17	7.0	7.1	UCSC WAHO	WAHO	DS		16.9	000	0.3695	090	0.6380	090	0.2700	VRT	0.2700
Mammoth Lakes (A)	1980.05.25	6.2	6.1	CDMG 324	A-CONVICT CREEK	DS		1.0	090	0.4165	180	0.4416	180	0.3900	VRT	0.3900
Mammoth Lakes (A)	1980.05.25	6.2	6.1	CDMG 444	A-LONG VALLEY DAM	R		3.2	000	0.4295	090	0.2708	090	0.1200	VRT	0.1200
Mammoth Lakes (A)	1980.05.25	6.2	6.1	CDMG 490	A-MAMMOTH LAKES H.S.	SS		3.2	000	0.2391	270	0.3211	270	0.2500	VRT	0.2500
Mammoth Lakes (B)	1980.05.25	5.7	6.0	CDMG 324	B-CONVICT CREEK	DS		1.0	090	0.1605	180	0.1783	180	0.1300	VRT	0.1300
Mammoth Lakes (B)	1980.05.25	5.7	6.0	CDMG 444	B-LONG VALLEY DAM	R		3.2	000	0.1942	090	0.0652	090	0.0400	VRT	0.0400
Mammoth Lakes (B)	1980.05.25	5.7	6.0	CDMG 490	B-MAMMOTH LAKES H.S.	SS		3.2	000	0.4408	270	0.3895	270	0.2600	VRT	0.2600
Mammoth Lakes (C)	1980.05.25	6.0	6.1	CDMG 324	C-CONVICT CREEK	DS		1.0	090	0.1327	180	0.0993	180	0.0500	VRT	0.0500
Mammoth Lakes (C)	1980.05.25	6.0	6.1	CDMG 444	C-LONG VALLEY DAM	R		3.2	000	0.1096	090	0.0709	090	0.0300	VRT	0.0300
Mammoth Lakes (D)	1980.05.27	6.0	6.2	CDMG 324	D-CONVICT CREEK	DS		1.0	090	0.2658	180	0.3156	180	0.1900	VRT	0.1900
Mammoth Lakes (D)	1980.05.27	6.0	6.2	CDMG 325	D-BENTON	DS		33.0	000	0.1745	270	0.1093	270	0.0600	VRT	0.0600
Mammoth Lakes (D)	1980.05.27	6.0	6.2	CDMG 444	D-LONG VALLEY DAM	R		3.2	000	0.9214	090	0.4082	090	0.3100	VRT	0.3100
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1001	APEEL 1-FOSTER CITY	DS	55.0	47.7	040	0.0427	310	0.0645	310	0.0200	VRT	0.0200
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1180	APEEL 1E HAYWARD	DS	52.0	47.7	000	0.0406	090	0.0267	090	0.0100	VRT	0.0100
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1250	GILROY GAVILAN COLL.	SS	39.0	15.9	067	0.1152	337	0.0946	337	0.0800	VRT	0.0800
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1251	CORRALITOS	SS	32.0	24.1	220	0.0811	310	0.1091	310	0.0500	VRT	0.0500
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1256	LOS BANOS	DS	79.0		090	0.0509	180	0.0574	180	0.0100	VRT	0.0100
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1299	FREMONT MISSION SJ	SS	36.0	31.6	075	0.0252	345	0.0208	345	0.0200	VRT	0.0200
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1301	AGNEWS STATE HOSPITAL	DS	26.0	23.0	240	0.0320	330	0.0316	330	0.0200	VRT	0.0200
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1376	CAPITOLA	DS	45.0	39.2	042	0.0991	132	0.1416	132	0.0400	VRT	0.0400
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1377	SAN JUAN BAUTISTA	SS	54.0	29.9	213	0.0436	303	0.0358	303	0.0500	VRT	0.0500
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1384	SANTA CRUZ UCSC LICK OBS	R	49.0	47.3	050	0.0391	320	0.0758	320	0.0300	VRT	0.0300
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1408	GILROY #1	R	39.0	16.0	230	0.0693	320	0.0983	320	0.0900	VRT	0.0900
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1409	GILROY #2	DS	39.0	14.9	000	0.1621	090	0.2130	090	0.5800	VRT	0.5800
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1410	GILROY #3	DS	39.0	14.4	000	0.1948	090	0.2010	090	0.4000	VRT	0.4000
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1411	GILROY #4	DS	37.0	12.6	270	0.2245	360	0.3490	360	0.4100	VRT	0.4100
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1413	GILROY #6	R	37.0	11.6	000	0.2228	090	0.2922	090	0.1100	VRT	0.1100
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1422	HALLS VALLEY	DS	4.0	3.2	150	0.1565	240	0.3121	240	0.1100	VRT	0.1100
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1445	COYOTE LAKE DAM (CREEK)	R	25.0	3.2	195	0.7112	285	1.2998	285	0.3900	VRT	0.3900

Table 2.C-1 (continued)

Strong Motion Data Base used in Analysis of Site Amplification and Vertical/Horizontal Ratios

Earthquake	Date	M	M _{INS}	ST. NO.	Station	SC	ED (km)	CD (km)	Comp	PGA (g)	Comp	PGA (g)	Comp	PGA (g)
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1455	SF INTERN. AIRPORT	DS	72.0		050	0.0478	320	0.0478	VRT	0.0200
Morgan Hill, CA	1984.04.24	6.2	6.2	CDMG 1595	GILROY #7 MANTELLI RANCH	SS	38.0	13.7	000	0.1912	090	0.1131	VRT	0.4300
Morgan Hill, CA	1984.04.24	6.2	6.2	USGS 1652	ANDERSON DAM (DUNSTRM)	DS		3.8	250	0.4230	340	0.2890	VRT	0.2000
Morgan Hill, CA	1984.04.24	6.2	6.2	USGS 1655L	SAN JUSTO DAM L. ABUT.	R		34.4	270	0.0810	360	0.0697	VRT	0.0300
Morgan Hill, CA	1984.04.24	6.2	6.2	USGS 1655R	SAN JUSTO DAM R. ABUT.	R		32.2	270	0.0783	360	0.0596	VRT	0.0400
Morgan Hill, CA	1984.04.24	6.2	6.2	USGS 1656	HOLLISTER DIFF. ARRAY	DS		27.9	165	0.0924	255	0.0998	VRT	0.2200
Nahanni, CAN	1985.12.23	6.9		CANA ST1	SITE 1	R	8.0	6.0	010	0.9776	280	1.0969	VRT	2.0900
Nahanni, CAN	1985.12.23	6.9		CANA ST3	SITE 3	R	20.0	16.0	270	0.1477	360	0.1390	VRT	0.1400
N. Palm Spr., CA	1986.07.08	6.2	5.9	CDMG 123	RIVERSIDE AIRPORT	DS		72.4	180	0.0509	270	0.0389	VRT	0.0200
N. Palm Spr., CA	1986.07.08	6.2	5.9	CDMG 168	PUERTA LA CRUZ USFS SB	SS		73.3	258	0.0763	348	0.0567	VRT	0.0400
Parkfield, CA	1966.06.27	6.1	5.6	CDMG 1013	CHOLAME 2	DS	28.9	0.1	065	0.4758	355	0.3670	VRT	0.2550
Parkfield, CA	1966.06.27	6.1	5.6	CDMG 1014	CHOLAME 5	SS	30.2	5.3	085	0.4416	320	0.2726	VRT	0.1400
Parkfield, CA	1966.06.27	6.1	5.6	CDMG 1015	CHOLAME 8	SS	31.4	9.2	050	0.2462	320	0.2726	VRT	0.1200
Parkfield, CA	1966.06.27	6.1	5.6	CDMG 1016	CHOLAME 12	SS	33.7	14.7	050	0.0594	320	0.0633	VRT	0.0500
Parkfield, CA	1966.06.27	6.1	5.6	CDMG 1438	TENBLOR PRE-1969	R	38.4	9.9	205	0.3574	295	0.2718	VRT	0.1400
Santa Barbara, CA	1978.08.13	6.0	6.0	USGS 106	CACHUMA DAM TOE	R		25.9	250	0.0721	340	0.0370	VRT	0.0200
San Fernando, CA	1971.02.09	6.6	6.4	CIT 103	ANZA POST OFFICE	DS	185.7	169.0	045	0.0267	315	0.0365	VRT	0.0200
San Fernando, CA	1971.02.09	6.6	6.4	LAF 104	SANTA ANITA DAM	R	43.2	27.0	003	0.1507	273	0.2122	VRT	0.0600
San Fernando, CA	1971.02.09	6.6	6.4	ACOE 108	CARBON CANYON DAM	R	79.8		130	0.0695	220	0.0706	VRT	0.0400
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 110	CASTAIC RIDGE	R	28.5	24.9	021	0.3239	291	0.2678	VRT	0.1700
San Fernando, CA	1971.02.09	6.6	6.4	CIT 111	CEDAR SPR. MILLER CNYN	R	185.7	169.0	050	0.0198	185	0.0152	VRT	0.0100
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 121	FAIRMONT DAM	R	32.6	29.1	056	0.0705	326	0.1051	VRT	0.0400
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 123	HEMET FIRE ST	DS	151.4	136.0	135	0.0371	225	0.0420	VRT	0.0300
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 125	LAKE HUGHES 1	SS	29.2	25.8	021	0.1445	111	0.1103	VRT	0.1000
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 126	LAKE HUGHES 4	R	27.3	24.2	111	0.1922	201	0.1527	VRT	0.1600
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 127	LAKE HUGHES 9	R	26.3	23.5	021	0.1573	291	0.1338	VRT	0.0900
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 128	LAKE HUGHES 12	R	23.0	20.3	021	0.3656	291	0.2831	VRT	0.1700
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 1319	TEHACHAPI FIRE	R	70.2	68.0	090	0.0527	180	0.0250	VRT	0.0400
San Fernando, CA	1971.02.09	6.6	6.4	USGS 135	LA-HOLLYWOOD LOT	DS	36.9	21.2	090	0.2098	180	0.1743	VRT	0.1400
San Fernando, CA	1971.02.09	6.6	6.4	USGS 141	LA GRIFFITH PARK OBS	R	33.8	17.4	180	0.1874	270	0.1838	VRT	0.1400
San Fernando, CA	1971.02.09	6.6	6.4	CDMR 269	PEARBLOSSOM PUMP	DS	45.2	38.9	000	0.1018	270	0.1357	VRT	0.0500
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 272	PORT HUENUME-NAVY	DS	80.2	63.0	180	0.0267	270	0.0252	VRT	0.0100
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 278	PUDDINGST. DAM	R	65.0	50.4	055	0.0738	325	0.0571	VRT	0.0400
San Fernando, CA	1971.02.09	6.6	6.4	CDMG 279	PAGOIMA DAM	R	8.7	2.8	164	1.2260	254	1.1595	VRT	0.7000
San Fernando, CA	1971.02.09	6.6	6.4	ACOE 287	UPLAND-SAN ANTONIO DAM	R	74.9		015	0.0580	285	0.0788	VRT	0.0300
San Fernando, CA	1971.02.09	6.6	6.4	USGS 465	S. JUAN CAPITRANO	DS	122.5	104.0	033	0.0434	303	0.0331	VRT	0.0200
Taiwan, SMART1 (5)	1981.01.29	5.7	6.3	TAIW C-00	SMART1 C-00	DS	30.0		180	0.1141	270	0.0964	VRT	0.0400
Taiwan, SMART1 (5)	1981.01.29	5.7	6.3	TAIW I-06	SMART1 I-06	DS		21.0	180	0.0768	270	0.0896	VRT	0.0300
Taiwan, SMART1 (5)	1981.01.29	5.7	6.3	TAIW I-12	SMART1 I-12	DS		21.0	180	0.1134	270	0.1403	VRT	0.0600
Taiwan, SMART1 (5)	1981.01.29	5.7	6.3	TAIW M-01	SMART1 M-01	DS		21.0	180	0.1776	270	0.0822	VRT	0.1000
Taiwan, SMART1 (5)	1981.01.29	5.7	6.3	TAIW M-07	SMART1 M-07	DS		21.0	180	0.1090	270	0.1109	VRT	0.0500
Taiwan, SMART1 (5)	1981.01.29	5.7	6.3	TAIW O-01	SMART1 O-01	DS		21.0	180	0.1152	270	0.0895	VRT	0.0300
Taiwan, SMART1 (5)	1981.01.29	5.7	6.3	TAIW O-07	SMART1 O-07	DS		21.0	180	0.0804	270	0.0864	VRT	0.0300
Taiwan, SMART1 (25)	1983.09.21	6.5	6.8	TAIW C00	SMART1 C00	DS	68.0	83.0	180	0.0284	270	0.0277	VRT	0.0100

Table 2.C-1 (continued)

Strong Motion Data Base used in Analysis of Site Amplification and Vertical/Horizontal Ratios

Earthquake	Date	M	M _{INS}	ST.NO.	Station	SC	ED(km)	CD(km)	Comp	PGA(g)	Comp	PGA(g)	Comp	PGA(g)	Comp
Taiwan, SMART1 (25)	1983.09.21	6.5	6.8	TAIW E01	SMART1 E01	SS	68.0	83.0	180	0.0311	270	0.0273	VRT	0.0100	0.0100
Taiwan, SMART1 (25)	1983.09.21	6.5	6.8	TAIW E02	SMART1 E02	R	68.0	83.0	180	0.0200	270	0.0200	VRT	0.0100	0.0100
Taiwan, SMART1 (25)	1983.09.21	6.5	6.8	TAIW I01	SMART1 I01	DS	68.0	83.0	180	0.0361	270	0.0241	VRT	0.0100	0.0100
Taiwan, SMART1 (25)	1983.09.21	6.5	6.8	TAIW I07	SMART1 I07	DS	68.0	83.0	180	0.0268	270	0.0347	VRT	0.0100	0.0100
Taiwan, SMART1 (25)	1983.09.21	6.5	6.8	TAIW M01	SMART1 M01	DS	68.0	83.0	180	0.0395	270	0.0199	VRT	0.0100	0.0100
Taiwan, SMART1 (25)	1983.09.21	6.5	6.8	TAIW M06	SMART1 M06	DS	68.0	83.0	180	0.0310	270	0.0195	VRT	0.0100	0.0100
Taiwan, SMART1 (25)	1983.09.21	6.5	6.8	TAIW O01	SMART1 O01	DS	68.0	83.0	180	0.0271	270	0.0279	VRT	0.0100	0.0100
Taiwan, SMART1 (25)	1983.09.21	6.5	6.8	TAIW O07	SMART1 O07	DS	68.0	83.0	180	0.0266	270	0.0247	VRT	0.0100	0.0100
Taiwan, SMART1 (33)	1985.06.12	5.2	6.5	TAIW C00	SMART1 C00	DS	45.0	45.0	180	0.0531	270	0.0789	VRT	0.0200	0.0200
Taiwan, SMART1 (33)	1985.06.12	5.2	6.5	TAIW I01	SMART1 I01	DS	45.0	45.0	180	0.0739	270	0.1449	VRT	0.0200	0.0200
Taiwan, SMART1 (33)	1985.06.12	5.2	6.5	TAIW I07	SMART1 I07	DS	45.0	45.0	180	0.0365	270	0.0497	VRT	0.0100	0.0100
Taiwan, SMART1 (33)	1985.06.12	5.2	6.5	TAIW M01	SMART1 M01	DS	45.0	45.0	180	0.0479	270	0.0302	VRT	0.0200	0.0200
Taiwan, SMART1 (33)	1985.06.12	5.2	6.5	TAIW M07	SMART1 M07	DS	45.0	45.0	180	0.0463	270	0.0956	VRT	0.0200	0.0200
Taiwan, SMART1 (33)	1985.06.12	5.2	6.5	TAIW O01	SMART1 O01	DS	45.0	45.0	180	0.0500	270	0.0622	VRT	0.0200	0.0200
Taiwan, SMART1 (33)	1985.06.12	5.2	6.5	TAIW O07	SMART1 O07	DS	45.0	45.0	180	0.0486	270	0.0641	VRT	0.0200	0.0200
Taiwan, SMART1 (40)	1986.05.20	6.4	6.5	TAIW C00	SMART1 C00	DS	67.9	64.0	000	0.2324	090	0.1719	VRT	0.0400	0.0400
Taiwan, SMART1 (40)	1986.05.20	6.4	6.5	TAIW E01	SMART1 E01	SS	64.0	64.0	000	0.1833	090	0.2030	VRT	0.0500	0.0500
Taiwan, SMART1 (40)	1986.05.20	6.4	6.5	TAIW I01	SMART1 I01	DS	64.0	64.0	000	0.1754	090	0.1834	VRT	0.0400	0.0400
Taiwan, SMART1 (40)	1986.05.20	6.4	6.5	TAIW I07	SMART1 I07	DS	64.0	64.0	000	0.1502	090	0.1642	VRT	0.0400	0.0400
Taiwan, SMART1 (40)	1986.05.20	6.4	6.5	TAIW M01	SMART1 M01	DS	64.0	64.0	000	0.1725	090	0.1563	VRT	0.0400	0.0400
Taiwan, SMART1 (40)	1986.05.20	6.4	6.5	TAIW M07	SMART1 M07	DS	64.0	64.0	000	0.2537	090	0.1817	VRT	0.0400	0.0400
Taiwan, SMART1 (40)	1986.05.20	6.4	6.5	TAIW O01	SMART1 O01	DS	64.0	64.0	000	0.1598	090	0.1064	VRT	0.0300	0.0300
Taiwan, SMART1 (40)	1986.05.20	6.4	6.5	TAIW O07	SMART1 O07	DS	64.0	64.0	000	0.1638	090	0.1590	VRT	0.0700	0.0700
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW C00	SMART1 C00	DS	79.3	39.0	000	0.1533	090	0.1216	VRT	0.0700	0.0700
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW E01	SMART1 E01	SS	39.0	39.0	000	0.1894	090	0.1592	VRT	0.0800	0.0800
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW E02	SMART1 E02	R	39.0	39.0	000	0.1427	090	0.1362	VRT	0.0500	0.0500
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW I01	SMART1 I01	DS	39.0	39.0	000	0.1407	090	0.1324	VRT	0.0800	0.0800
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW I07	SMART1 I07	DS	39.0	39.0	000	0.1216	090	0.1179	VRT	0.0900	0.0900
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW M01	SMART1 M01	DS	39.0	39.0	000	0.1413	090	0.1187	VRT	0.0800	0.0800
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW M07	SMART1 M07	DS	39.0	39.0	000	0.1597	090	0.1556	VRT	0.1100	0.1100
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW O01	SMART1 O01	DS	39.0	39.0	000	0.1737	090	0.1260	VRT	0.0600	0.0600
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW O02	SMART1 O02	DS	39.0	39.0	000	0.2422	090	0.1605	VRT	0.0600	0.0600
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW O04	SMART1 O04	DS	39.0	39.0	000	0.1629	090	0.1259	VRT	0.0900	0.0900
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW O06	SMART1 O06	DS	39.0	39.0	000	0.1903	090	0.1710	VRT	0.0900	0.0900
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW O07	SMART1 O07	DS	39.0	39.0	000	0.1645	090	0.1540	VRT	0.1000	0.1000
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW O08	SMART1 O08	DS	39.0	39.0	000	0.1639	090	0.1423	VRT	0.1100	0.1100
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW O12	SMART1 O12	DS	39.0	39.0	000	0.1595	090	0.1399	VRT	0.0700	0.0700
Taiwan, SMART1 (45)	1986.11.14	7.8	7.0	TAIW O10	SMART1 O10	DS	39.0	39.0	000	0.1159	090	0.1482	VRT	0.0600	0.0600
Spitak, ARMENIA	1988.12.07	7.0		ARME GUK	GUKASTAN	R	35.0	30.0	000	0.1992	090	0.1753	VRT	0.1200	0.1200
Sup.Hills, CA (A)	1987.11.23	6.3	5.8	USGS 5210	WILDLIFE LIQUEF. ARRAY	DS	22.9	24.7	090	0.1320	360	0.1336	VRT	0.1900	0.1900
Sup.Hills, CA (B)	1987.11.24	6.7	6.6	USGS 5210	WILDLIFE LIQUEF. ARRAY	DS	24.5	24.4	090	0.1810	360	0.2072	VRT	0.4100	0.4100
Tabas, IRAN	1978.09.16	7.4		IRAN DAY	DAYHOOK	R	17.0	17.0	HR1	0.3282	HR2	0.4061	VRT	0.1800	0.1800
Tabas, IRAN	1978.09.16	7.4		IRAN TAB	TABAS	SS	3.0	3.0	HR1	0.8618	HR2	0.8518	VRT	0.6500	0.6500
Mexicali Valley, MX	1980.06.09	6.4	6.1	UCSB 6604	CERRO PRIETO	R	31.0	8.5	045	0.6211	315	0.5873	VRT	0.3000	0.3000

Table 2C-1 (continued)
Strong Motion Data Base used in Analysis of Site Amplification and Vertical/Horizontal Ratios

Earthquake	Date	M	M	INS	ST. NO.	Station	SC	ED(km)	CD(km)	Comp	PGA(g)	Comp	PGA(g)	Comp	PGA(g)
Mexicali Valley, MX	1980.06.09	6.4	6.1	UCSB	6617	CUPAPAH	DS	20.7	20.7	085	0.0757			VRT	0.0674
Mexicali Valley, MX	1980.06.09	6.4	6.1	UCSB	6619	SAHOP CASA FL.	DS	30.4	30.4	010	0.1015	280	0.0684	VRT	0.0500
Mexicali Valley, MX	1980.06.09	6.4	6.1	UCSB	6621	CHIHUAHUA	DS	14.6	14.6	102	0.1495	192	0.0918	VRT	0.1000
Mexicali Valley, MX	1980.06.09	6.4	6.1	UCSB	6624	HOSPITAL SOTANO	DS		35.0	000	0.0445	270	0.0320	VRT	0.0200

CODES

INS Data source

ST.NO. Station number

SC Site Classification

R Rock
SS Shallow soil (<~20 m to rock)
DS Deep soil (>~20 m to rock)

ED Epicentral distance

CD Closest distance to fault rupture

COMP Component

PGA Peak ground acceleration

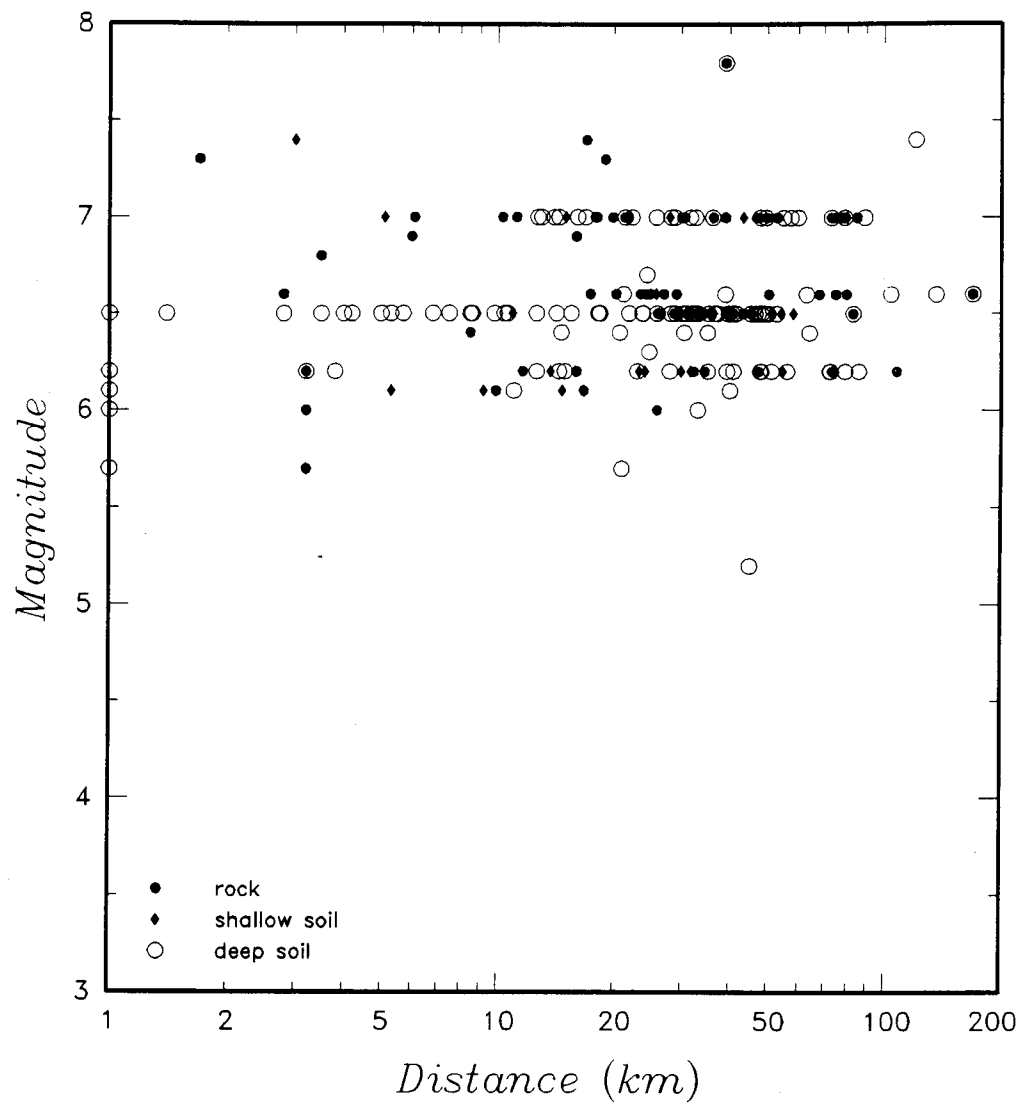


Figure 2.C-1
Magnitude-distance scattergram of strong motion data set

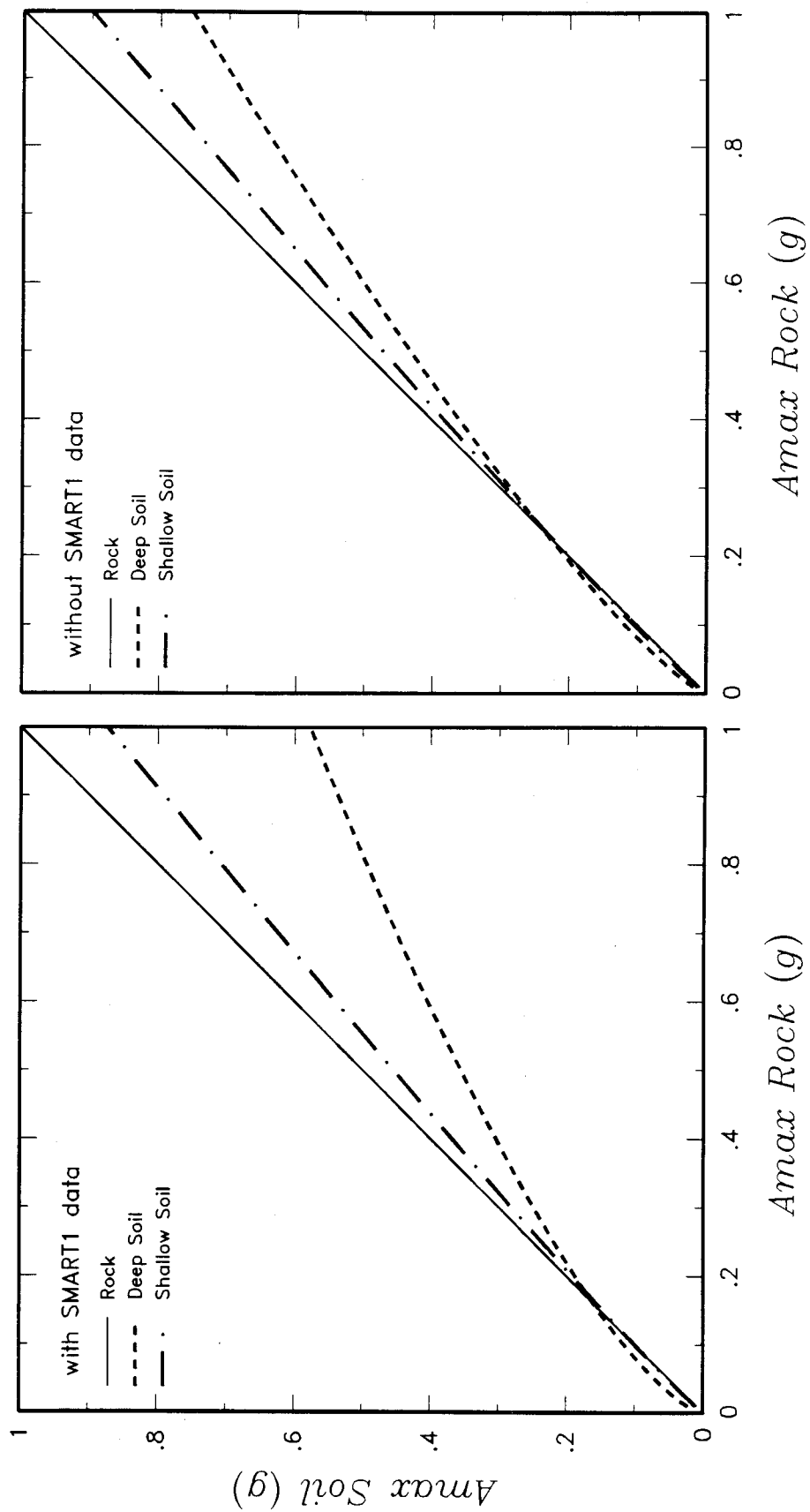


Figure 2.C-2
Soil/Rock peak acceleration ratios computed from data sets

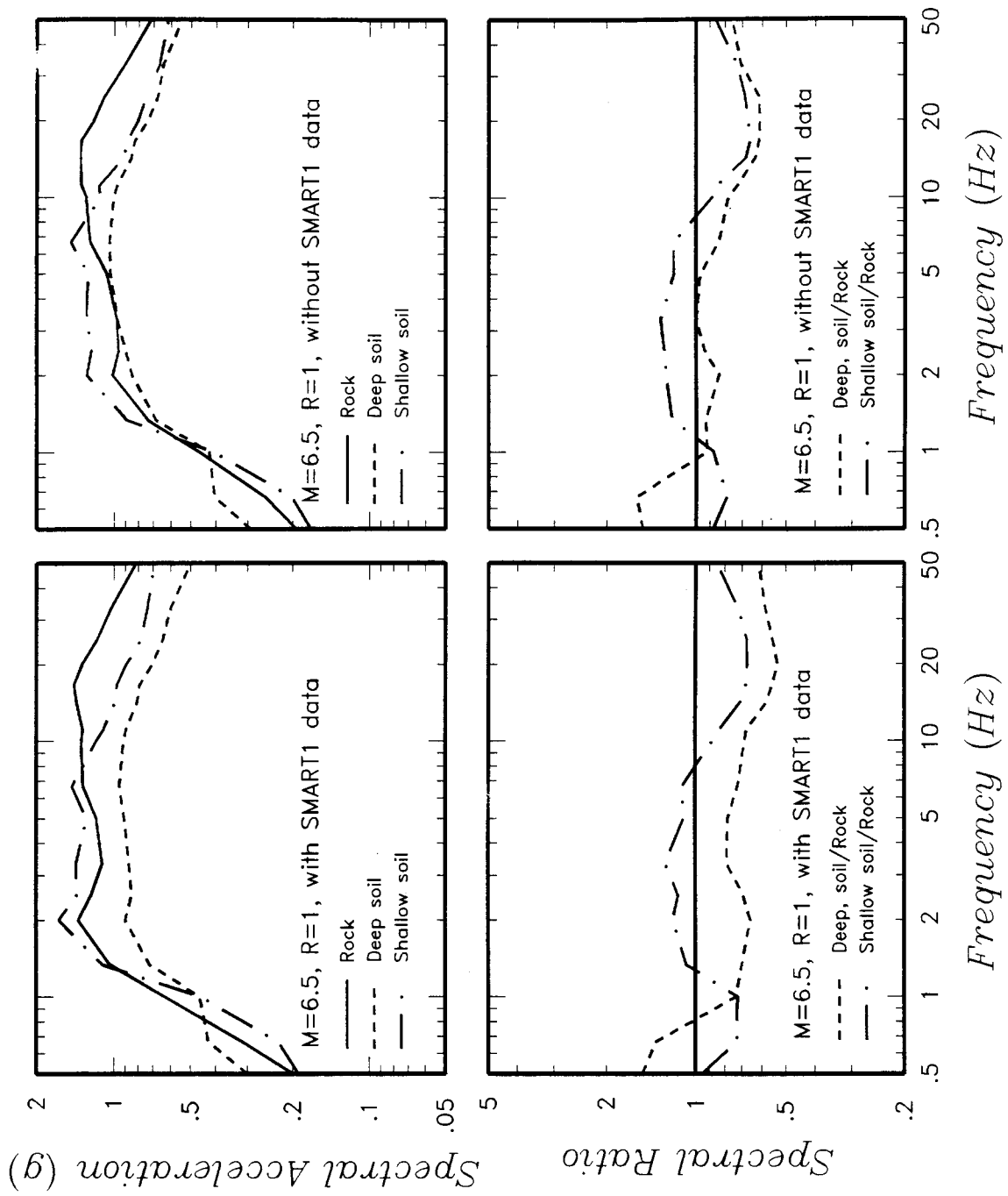


Figure 2.C-3

Response spectra and soil/rock spectral ratios computed for a distance of 1 km

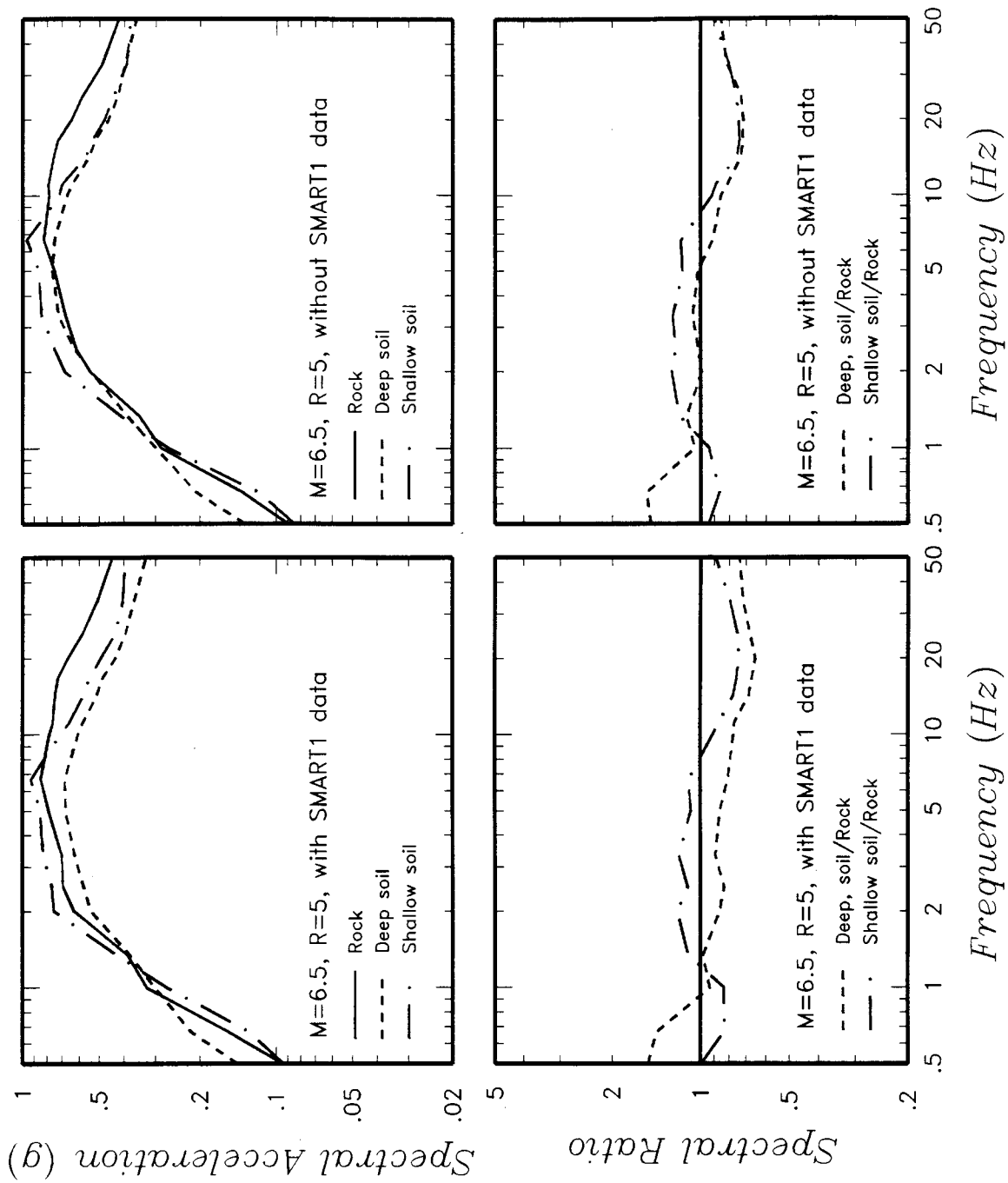


Figure 2.C-4
 Response spectra and soil/rock spectral ratios computed for a distance of 5 km

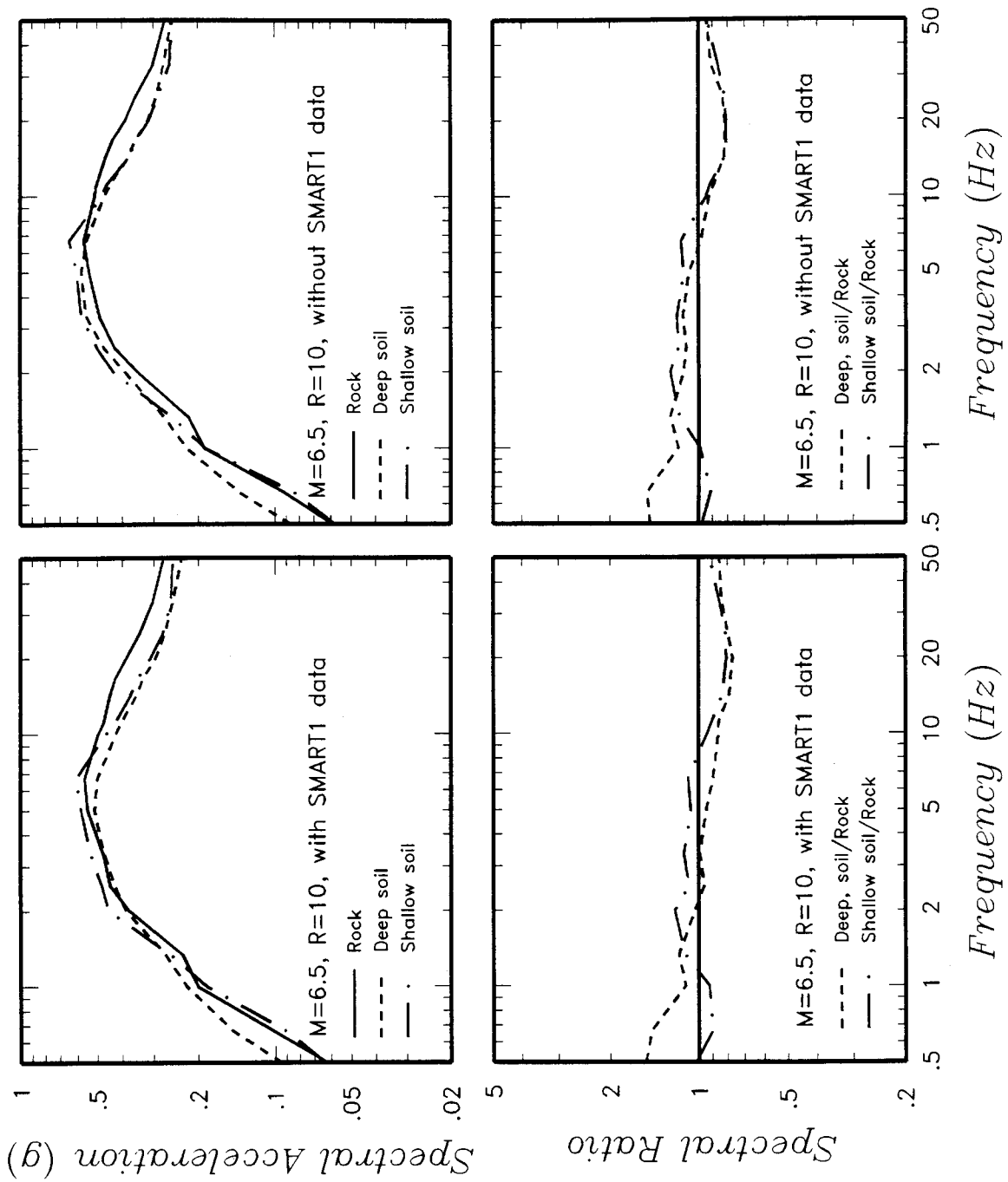


Figure 2.C-5
Response spectra and soil/rock spectral ratios computed for a distance of 10 km

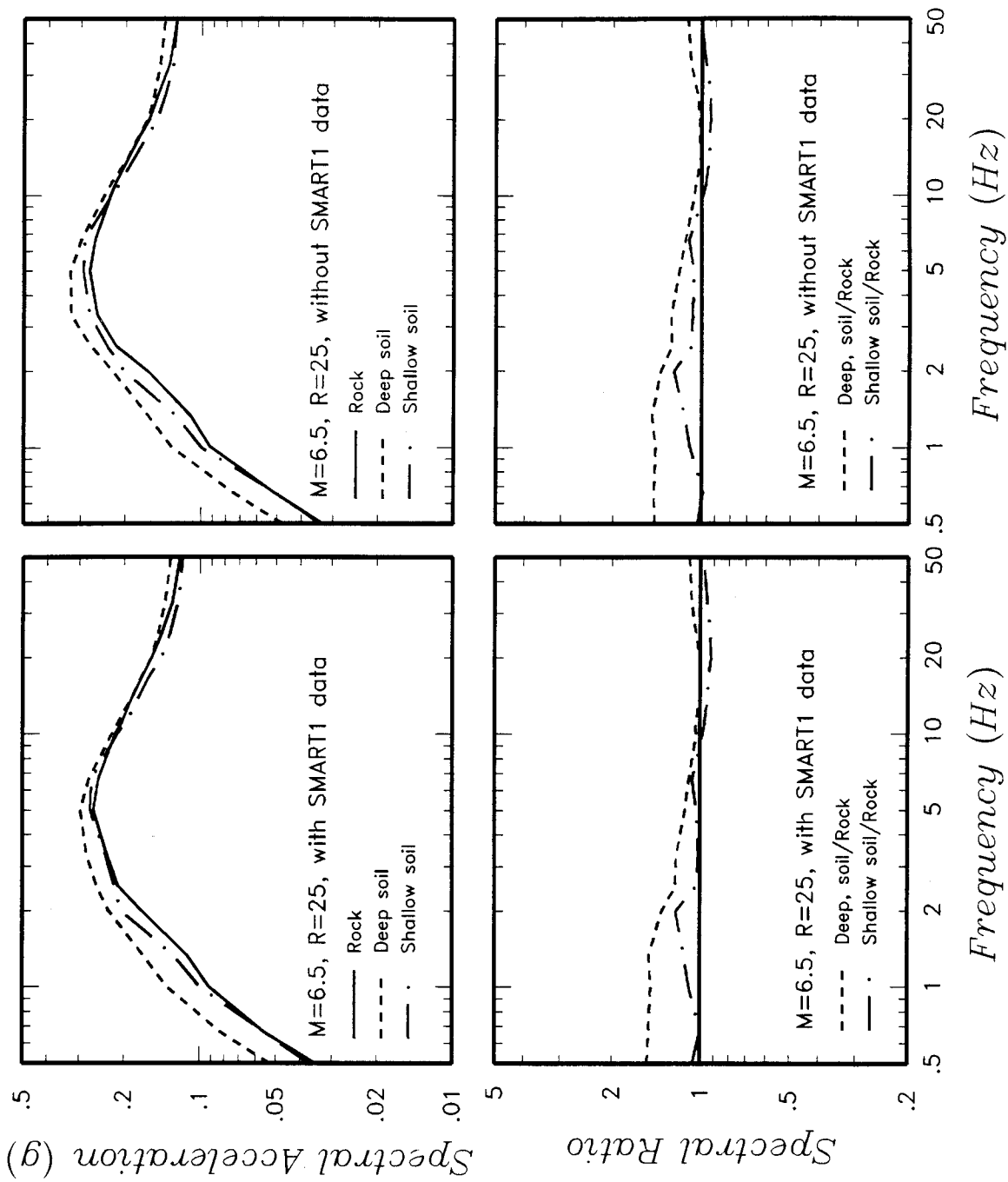


Figure 2.C-6
Response spectra and soil/rock spectral ratios computed for a distance of 25 km

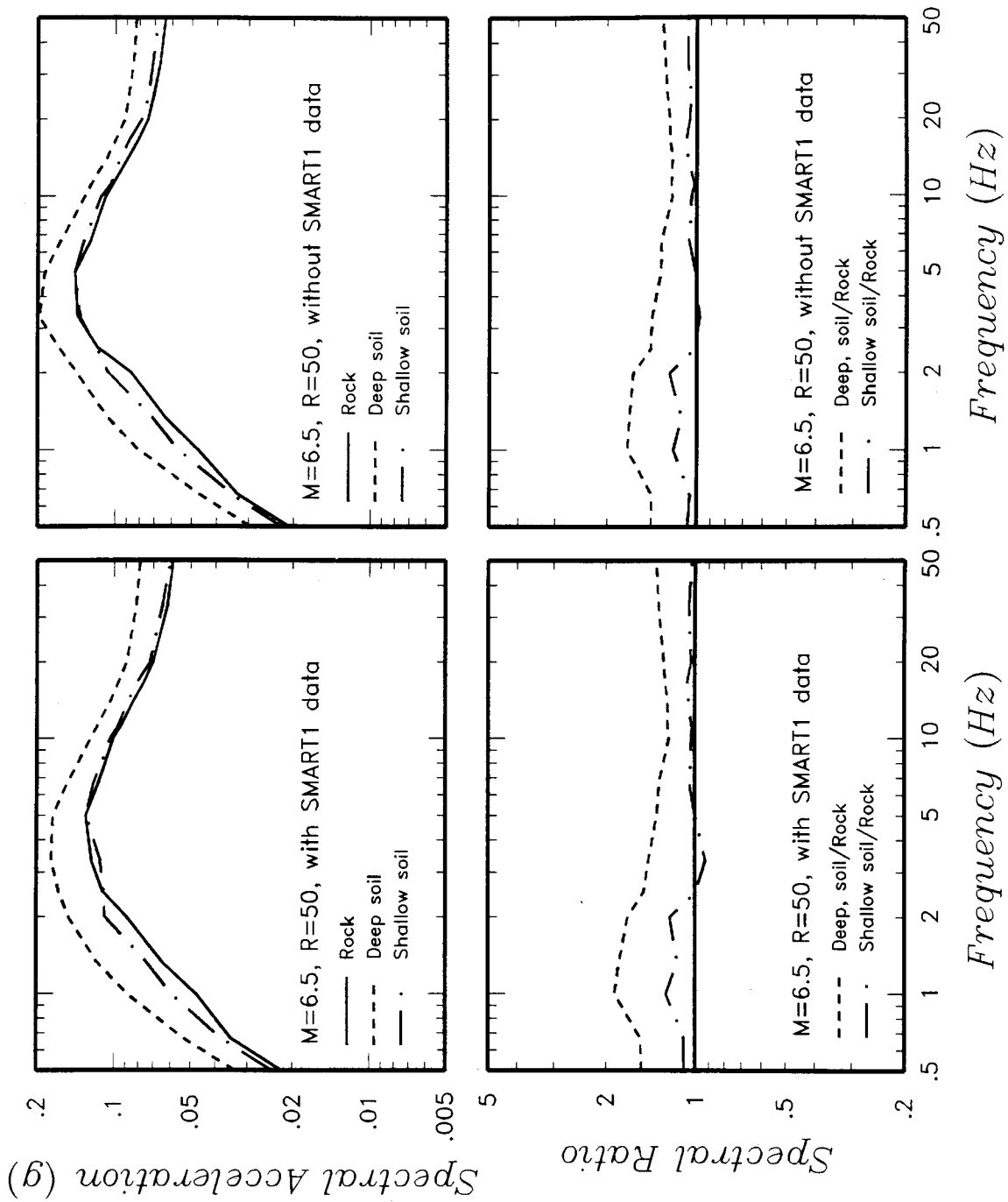


Figure 2.C-7
 Response spectra and soil/rock spectral ratios computed for a distance of 50 km

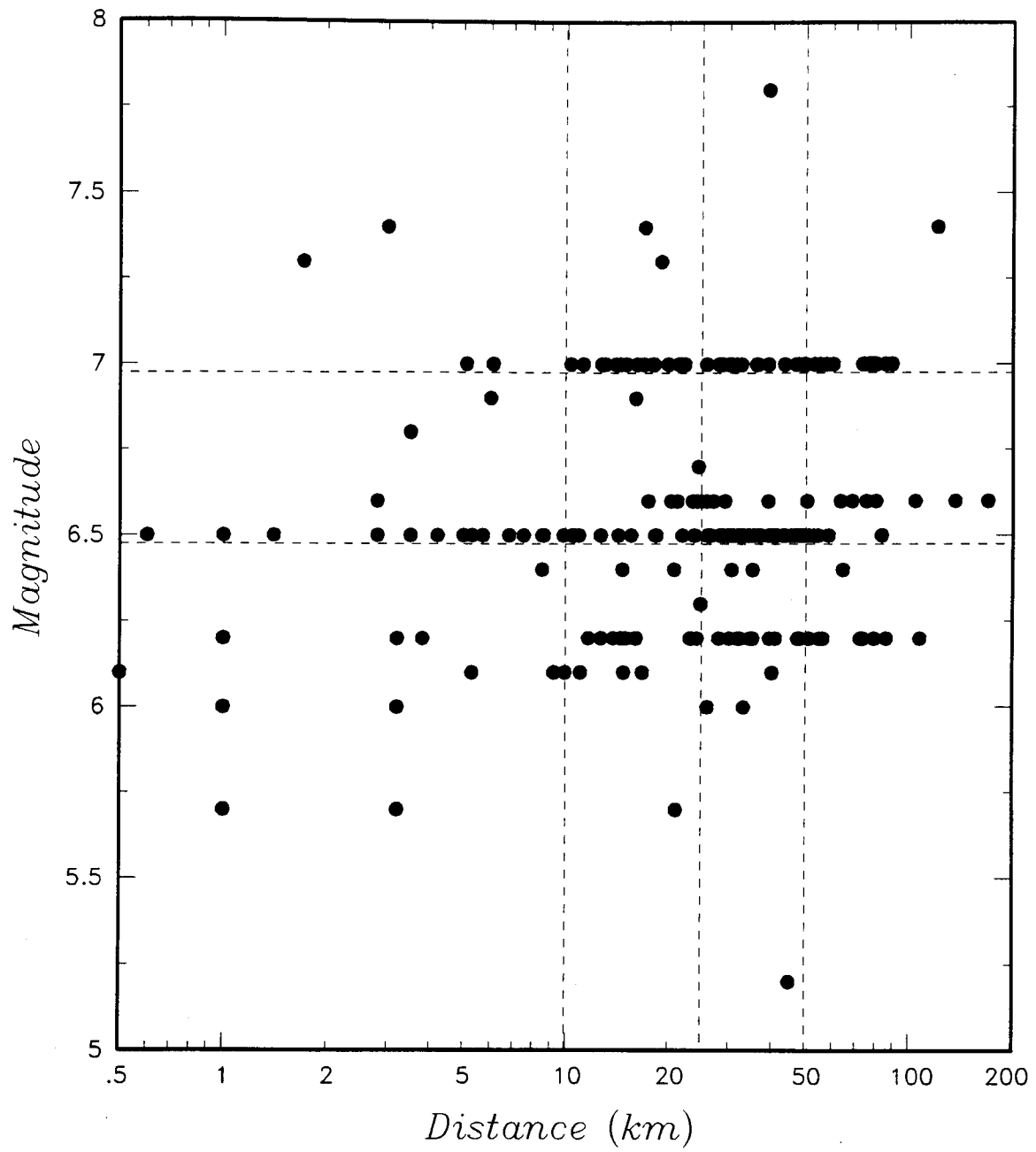


Figure 2.C-8
Magnitude-distance interval partitions of data used in statistical analysis of vertical/horizontal spectral ratios.

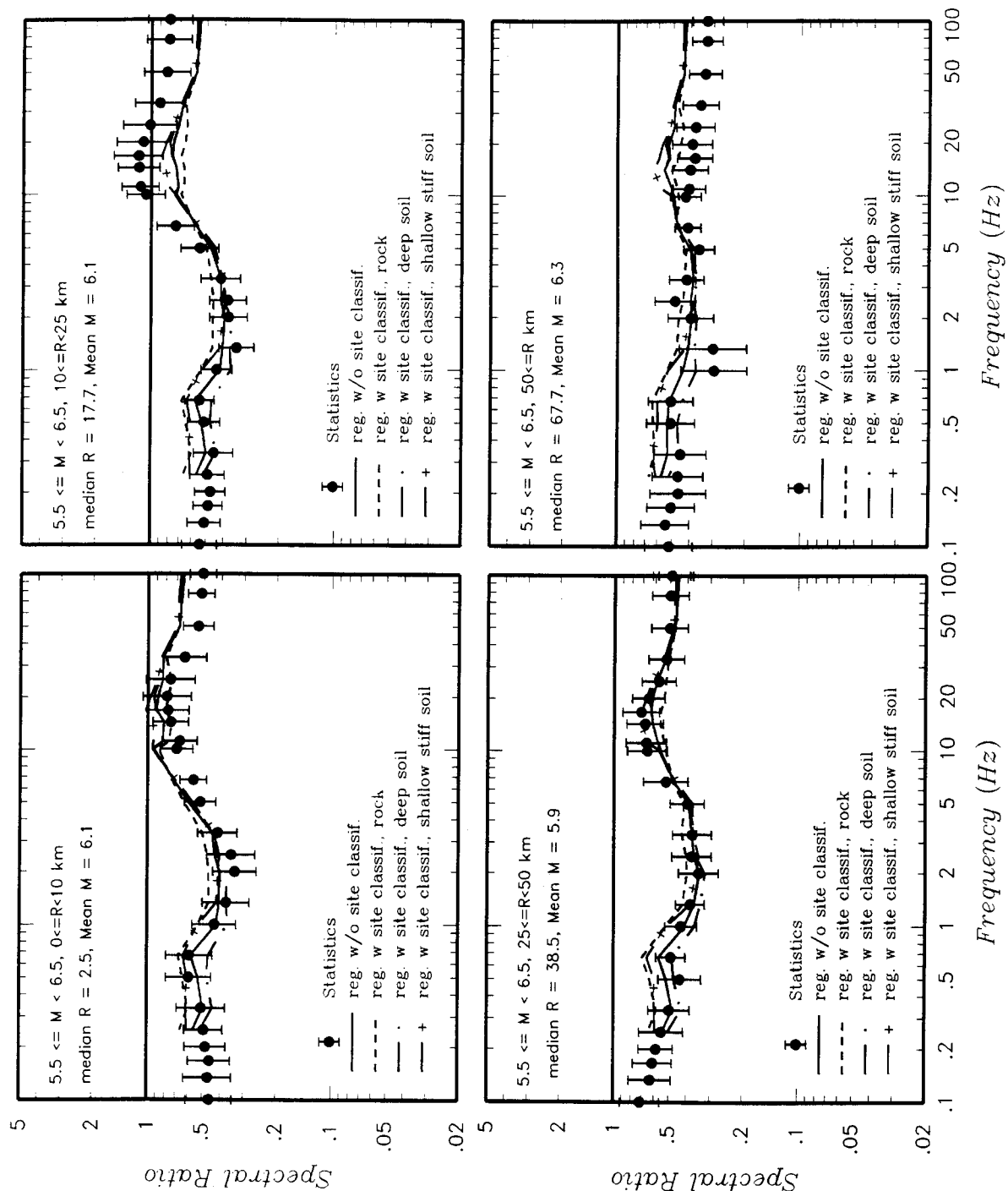


Figure 2.C-9a
Vertical/horizontal spectral ratios

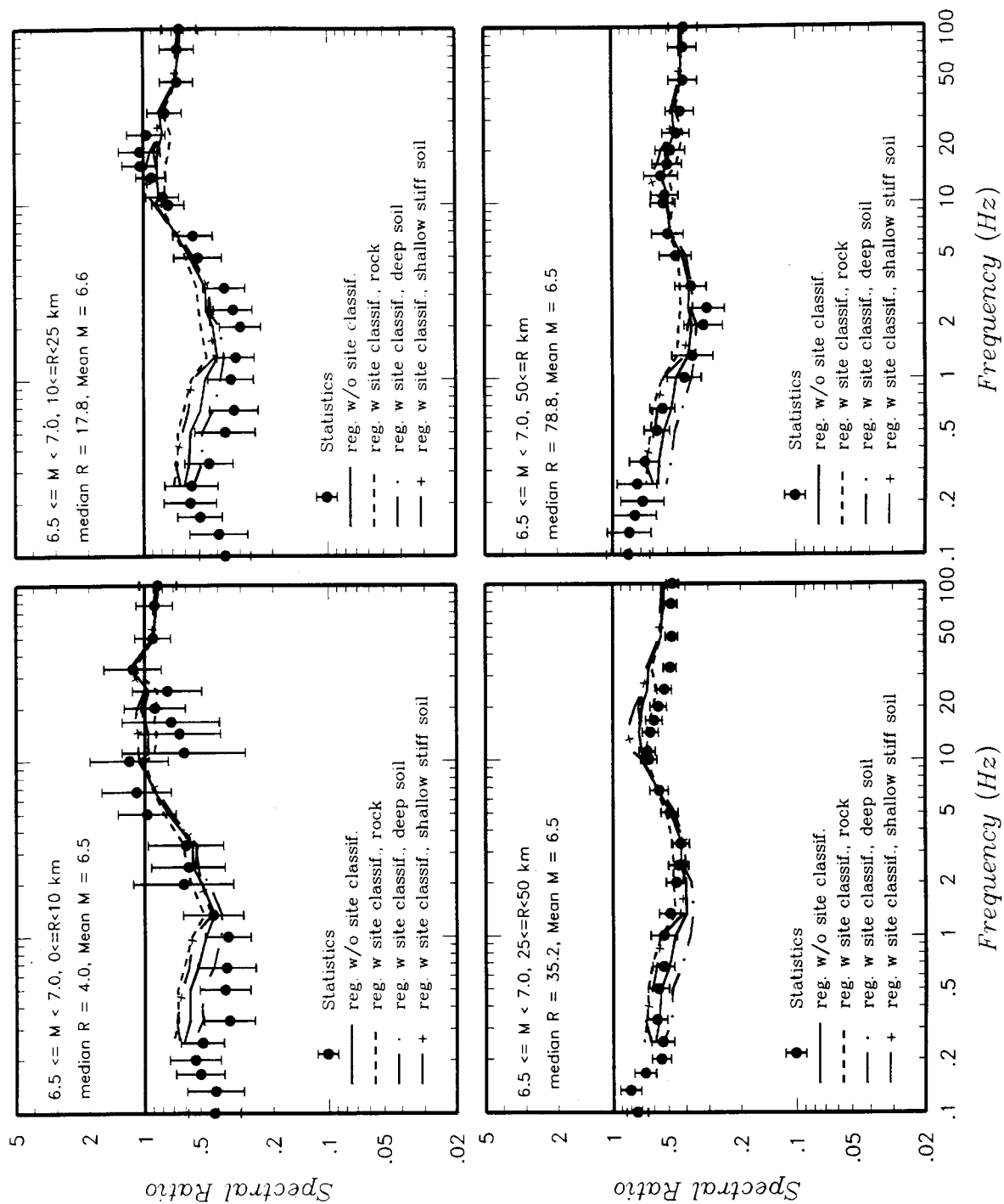


Figure 2.C-9b
 Vertical/horizontal spectral ratios

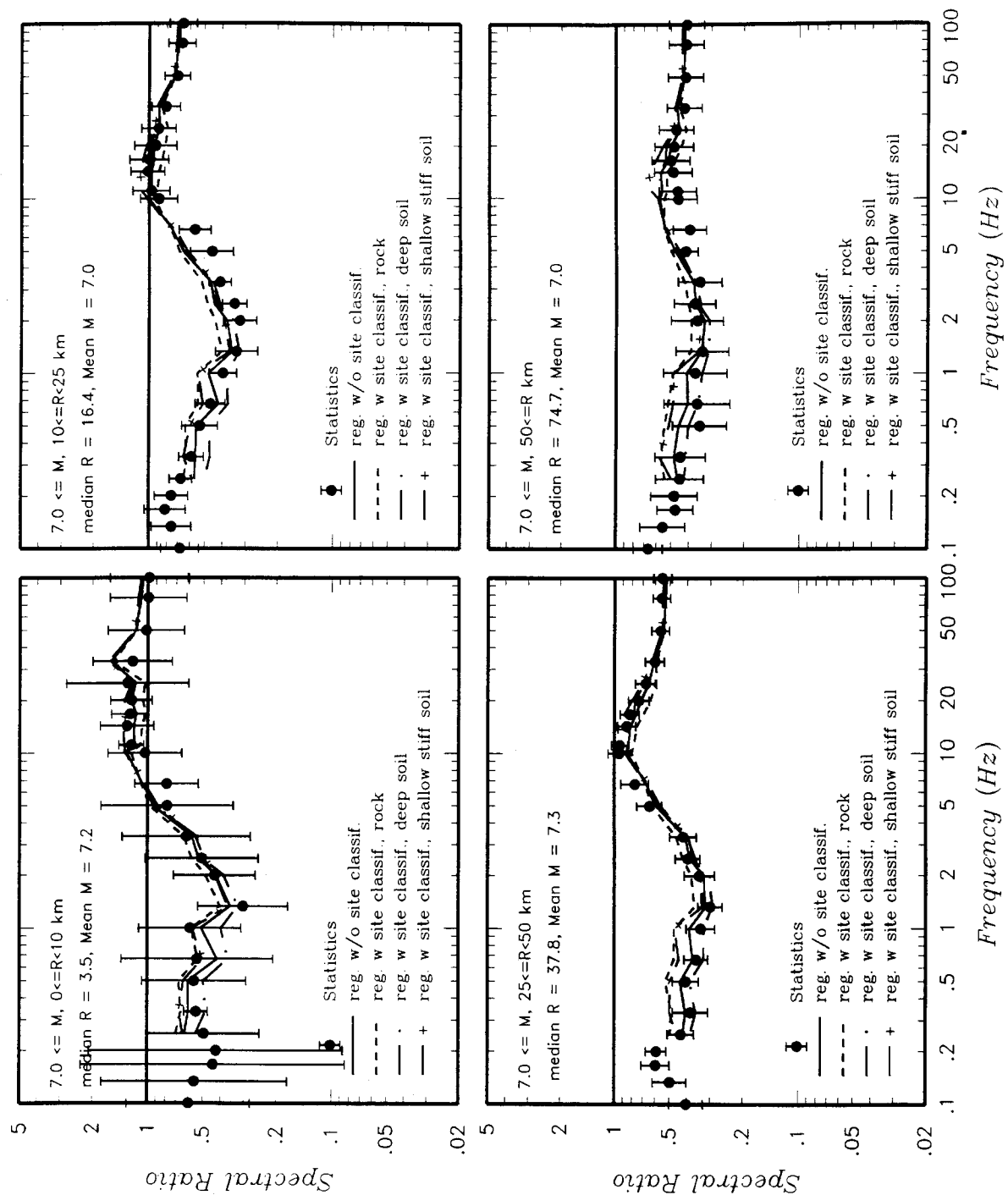


Figure 2.C-9c
Vertical/horizontal spectral ratios

APPENDIX 3.A

REVISION AND CALIBRATION OF THE OU AND HERRMANN METHOD

The simulation methodology described in Section 3.2 uses random process theory (RPT) to compute the response spectral values. Using RPT, the time domain peak amplitude, a_0 , and the Fourier amplitude spectrum, $a(f)$, are related through the signal duration, T . Ou and Herrmann (1990b) developed a method for incorporating crustal structure effects in a RPT method.

The Herrmann method (1992) is an extension of Ou and Herrmann (1990b) that increases the accuracy of the formulation making it compatible with asymptotic ray theory for plane layered crustal models. The extension accounts for reflection, transmission and free-surface coefficients, and some subcritical and all super-critical S-wave arrivals. In addition, a simplified technique is used to obtain average radiation pattern coefficients for the transverse peak motions for use with RPT. The results obtained using this method are compared to those from suites of time domain realizations of the ground motion using asymptotic ray theory and wave number integration.

3.A.1 Source Spectrum Model

In the Herrmann method, the acceleration source spectrum, $a(f)$, is modeled as a function of frequency, f , and distance, r by the equation

$$a(f,r) = \frac{\pi F M_0}{\beta_o^3 \rho_o} \left(\frac{\beta_o \rho_o}{\beta_R \rho_R} \right)^{1/2} G(r) \frac{f^2}{1 + (f/f_c)^2} \exp\left\{ \frac{-\pi f t}{Q(f,r)} \right\} P(f) H(f), \quad (3.A-1)$$

where M_0 is the seismic moment; F is constant accounting for the average radiation pattern and free-surface effect; ρ and β are the density and shear-wave velocity with subscripts o and R referring the source and receiver, respectively; $G(r)$ is the geometrical spreading term; f_c is the corner frequency; t is the travel time; $Q(f,r)$ is the whole path Q ; $P(f)$ is a high frequency truncation filter; and $H(f)$ is a general transfer function for a particular component of motion or instrument response.

In Section 3, the acceleration source spectrum for a point source (Eq. 3-1) is given by

$$a(f,r) = C M_0 A(f) \frac{1}{r} \frac{f^2}{1 + (f/f_c)^2} \exp\left\{ \frac{-\pi f r}{\beta_o Q(f)} \right\} P(f), \quad (3.A-2)$$

where C is a constant and $A(f)$ is a near-surface amplification factor. Comparing Eq. 3.A-1 and 3.A-2, the point source model uses $1/r$ for the geometrical spreading $G(r)$; it uses r/β_o for the travel time t ; and it uses the site amplification $A(f)$ in place of the impedance ratio between the source and receiver. The relation between the constants C and F is given by

$$C = \frac{\pi F}{\rho_o \beta_o^3}. \quad (3.A-3)$$

The Herrmann method is incorporated into the model in Section 3 by replacing the $1/r$ term with $G(r)$ and computing C from Eq. 3.A-3. The method for computing the $G(r)$ and F terms and the signal duration T is described in the following.

3.A.2 Estimation of Signal Duration

Based on an analysis of strong motion data from the Caltech Strong Motion catalog, Ou and Herrmann (1990a) found that defining the duration as the 5–75% time window of the normalized integral of the squared signal (acceleration), their procedure gave ground motion estimates that were consistent with the recorded data except at low frequencies ($f < 1\text{Hz}$).

Ou and Herrmann (1990b) developed a method to define the signal duration based on the arrival times and amplitudes of the sequence of ray arrivals. The procedure for estimating the signal duration from the sequence of arrivals has not changed in the Herrmann (1992) method, but the estimation of the ray amplitudes has changed.

3.A.3 Estimation of Ray Amplitudes

As a simplification, Ou and Herrmann (1990b) did not take into account the effect of transmission and reflection upon the ray amplitudes, rationalizing that the net transmission effect equaled unity and that supercritical reflections yielded unit amplitude. The objective of the Herrmann extension is to consider shear wave arrivals including subcritical as well as supercritical reflected arrivals, to correctly account for reflection and transmission along the ray, and to simplify the manner of computing the F factor.

If the geometrical spreading of each ray between the source and the receiver is given by $G_i(r)$, and if it is assumed that a random ground motion sequence is associated with each ray, then the total energy of the signal duration at distance r equals the sum of the energy in the individual arrivals. Combining the rays, the effective geometrical spreading is given by

$$G(r) = \left(\sum_i G_i^2(r) \right)^{1/2}. \quad (3.A-4)$$

In choosing the arrivals that contribute to this sum, the criteria that the ray time window represent the 5–75% cumulative signal squared window is used.

The effective $Q(f,r)$ was obtained by finding an appropriate operator to describe the signal content due to the arrivals as

$$\left(G(r) \exp\left\{ \frac{-\pi f T_0}{Q(f,r)} \right\} \right)^2 = \sum_i \left(G_i(r) \exp\left\{ \frac{-\pi f T_i}{Q_i} \right\} \right)^2, \quad (3.A-5)$$

where $\bar{Q}_i$ is the effective Q for the i^{th} ray, T_i is the travel time of the i^{th} ray, and T_0 is the mean travel time of the entire ray group.

The difference between the Herrmann extension and the Ou and Herrmann (1990b) method is the use of reflection and transmission coefficients. The Herrmann extension follows Cervený et al. (1977) in formulating the reflection and transmission coefficients. The ray amplitude due to a point source of shear waves is given by

$$U_i(f,r) = \frac{\pi R_{\theta\phi}}{\rho_S \beta_S^3} \left(\frac{\beta_S \rho_S}{\beta_R \rho_R} \right)^{1/2} G_i(r) S(f) \exp\left\{ \frac{-\pi f t}{Q(f,r)} \right\} P(f) H(f), \quad (3.A-6)$$

where

$$G_i(r) = \prod_{j=1}^N \left[\frac{\rho'(O_j) \beta'(O_j) \cos \theta_j}{\rho(O_j) \beta(O_j) \cos \theta_j} \right]^{1/2} \prod_{j=1}^N R_j \left[\frac{\sin \delta_s}{J(M)} \right]^{1/2}. \quad (3.A-7)$$

Eq. 3.A-6 accounts for the ray amplitude at a distance r due to an excitation $R_{\theta\phi}$ at a unit distance from the source. The θ_j and θ'_j are the incident and reflected/transmitted angles of incidence at each interface along the ray path. The R_j are the plane wave reflection and transmission coefficients for each interface interaction, and also partition the incidence ray into the free-surface amplitudes. The cosine terms account for curvature of the wavefront. The $J(M)$ term represents the ratio of the classical wavefront cross-sectional area at the receiver to that at the intersection of the ray tube with the unit sphere about the source. The $\sin \delta_s$ is proportional to the cross-sectional area on the sphere, where δ_s is the ray angle with respect to the vertical. The $J(M)$ term is easily computed from ray parameter—distance relations for plane layered media.

In effect, the Herrmann extension incorporates high frequency, asymptotic ray theory, incurring an error if the infinite ray group is truncated.

Because random process theory requires only the specification of the amplitude spectra and duration at the observation point, the complexity of the asymptotic ray description is replaced by a single $F G(r)$ term. The F and $G(r)$ terms are computed in two separate but not independent steps. The $G(r)$ is computed using Eq 3.A-4 and controls the average illumination of the receiver by the source at this distance. The F factor is obtained by computing the average spectra at the distance r and dividing by the $G(r)$ term.

$$F(r) = \frac{\bar{A}(r)}{G(r)}, \quad (3.A-8)$$

where $\bar{A}(r)$ is the median (log average) RMS spectrum for a range of focal mechanisms and the RMS spectrum is computed for the frequency band of 1 to 10 Hz.

$$\log \bar{A}(r) = \frac{1}{N_m} \sum_{j=1}^{N_m} \log \hat{A}_j(r), \quad (3.A-9)$$

where N_m is the number of focal mechanisms sampled and $\hat{A}_j(r)$ is the RMS spectrum for the j^{th} mechanism given by

$$\hat{A}_j(r) = \left(\frac{1}{N_f} \sum_{k=1}^{N_f} A_j(f_k, r) \right)^{1/2}, \quad (3.A-10)$$

where N_f is the number of frequencies between 1 and 10 Hz and

$$A(f, r) = \left| \sum_i (R_{\theta\phi})_i G_i(r) \exp\{-i2\pi f t_i\} \right|, \quad (3.A-11)$$

where t_i is the arrival time of the ray. In this analysis, only focal mechanism that have a pressure or tension axis within 22.5° of the horizontal are used. This constraint together with dip angle limits of 30 to 90° are representative of the range of well determined focal mechanisms in the EUS. The range of focal mechanisms sampled is given in Table 3.A-1.

3.A.4 Comparison with Time Domain Methods

The Oh and Herrmann procedure and the Herrmann extension make certain assumptions about the ground motion and its spectrum. Specifically, it is assumed that the time history associated with each ray is random enough that the multiple ray arrivals combine in an RMS sense in both the time and amplitude spectra domains. This assumption was used but not tested by Ou and Herrmann (1990b).

To test this assumption, synthetic seismograms are computed that use a time sequence that has the spectral shape given by $P(f)S(f)$ and which is suitably stochastic at high frequencies for random process theory to apply. Two time domain methods are also used to compute the time histories and peak ground motion values: high frequency asymptotic ray theory and wavenumber integration. The RPT model is based on asymptotic ray theory so the comparison of the time domain and RPT peak values will test the method for computing the mean spectra. Comparison to the wavenumber integration peak values will test the suitability of asymptotic ray theory to describe the time histories. These comparisons will also permit an evaluation of the consequences of replacing the $F(r)$ term by a distance independent constant.

Since RPT is based on a random time sequence, a test of the procedures described requires the use of a source time function that is suitably random. This is not a trivial exercise since this source time function must also have a source spectrum that is consistent with the ω^2 model. In addition, the displacement time history should be primarily positive to represent a unidirectional faulting process. One suitable source time function with a corner frequency of 0.19 Hz is shown in Figure 3.A-1. The sample interval is 0.005 sec and a Boore (1983) wavelet shape is used. The velocity and acceleration time histories are obtained by differentiating the displacement time history. The peak values are indicated as well as the RPT estimates of the mean peak value. The RPT estimate is based on the actual time history synthesized. In general, the actual ground motion peak values are smaller than the RPT estimates. Figure 3.A-2 shows the pseudo-acceleration response spectrum for the time series (symbols) compared to the desired value for the corner frequency used. The RPT response spectra estimates are greater than the time domain values by factors of 1.86 and 1.11 at frequencies of 5.0 and 1.0 Hz, respectively. This difference occurs because the selected source time function is not realistic in terms of its stochastic nature. In the following comparisons, we expect to see differences of this magnitude even if the wave propagation calculations were identical.

Simulations were performed for the Mid-continent and Gulf Coast models (see Section 5.1) for a focal depth of 10. For these comparisons, the effect of Q is ignored.

The F values for the Mid-continent crustal model at a depth of 10 km are shown in Figure 3.A-3 as a function of distance for the transverse component. The F factor increases with distance at distances less than 100 km. This occurs because only a single ray contributes to the ground motion at short distances and that on average for the range of focal mechanisms considered, the near vertical rays are somewhat lower in amplitude. At larger distances, the distance dependence is not as strong because many more rays are involved and a stable average over the focal sphere is obtained. The geometric mean F factors averaged over distances of 1 to 500 km for the transverse and radial components are 0.42 and 0.41, respectively.

The response spectra at 1 Hz and 5 Hz computed using the RPT method and the two time domain methods are compared in Figures 3.A-4a, b for the transverse component. The solid line is the RPT estimate, the dot-dashed line is the wavenumber integration result, and the short dashed line is the asymptotic ray theory result. The RPT estimates are expected to be about 11% larger at 1 Hz and 86% higher at 5 Hz due to the source time function used. The three methods show good agreement given these correction factors. At 1 Hz, the RPT estimate is conservative at moderate distances (30–70 km) while at 5 Hz, the RPT estimate is unconservative at distances greater than 100 km. The response at 1 Hz and 5 Hz using a distance independent F factor of 0.41 is shown in Figures 3.A-5a, b. The use of a distance independent F increases the conservatism of the RPT estimates at short distances. Similar results are obtained for other focal depths.

3.A.5 Conclusions

The wave propagation for the Herrmann extension to the Ou and Herrmann (1990b) method yields attenuation that is consistent with time domain methods. If distance independent F factors are used, then the Herrmann method leads to conservative estimates of the peak ground motion values at short distances in comparison to the time domain methods.

References

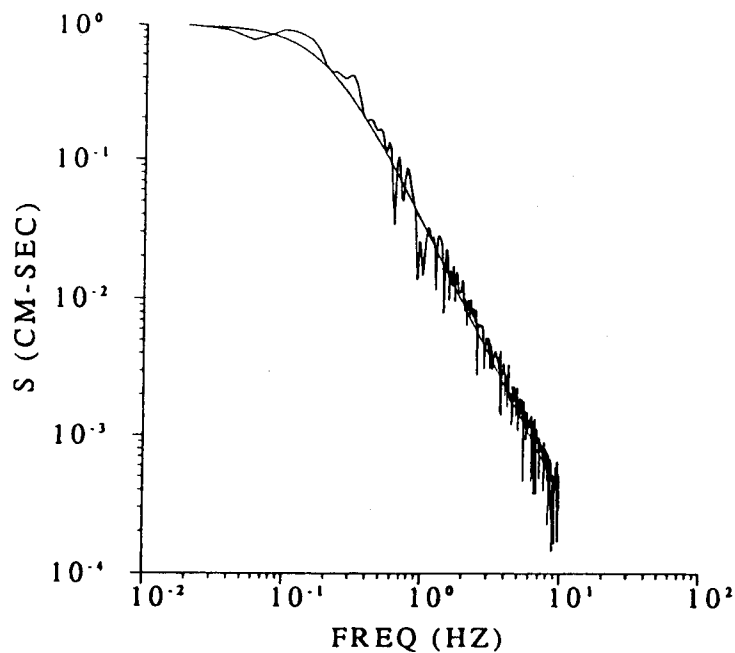
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Table 3.A-1

	Minimum	Maximum	Increment
Dip	30	90	15
Strike	0	170	10
Slip	-180	150	30

Table 3.A-2**Mid-continent Crustal Model**

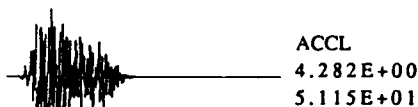
Thickness	α	β	ρ
(km)	(km/s)	(km/s)	(gm/cm³)
1	4.90	2.83	2.52
11	6.10	3.52	2.71
28	6.18	3.57	2.78
	8.00	4.62	3.35



FZ= 0.222 RPT_MEAN= 0.732
DUR[05,75]=1.850 XMAX=1.613 XMIN=0.197



FZ= 1.343 RPT_MEAN= 0.946
DUR[05,75]=4.500 XMAX=1.401 XMIN=0.670



FZ= 4.699 RPT_MEAN= 7.041
DUR[05,75]=6.150 XMAX=9.551 XMIN=5.516

Figure 3.A-1

Realization of time history having desired spectrum and duration. The corner frequency of the model is $f_c = 0.19$ Hz. The peak amplitude and duration of each plotted trace is shown. In addition, the spectra of each time history was used with RPT to estimate the frequency, FZ, from zero crossings, the estimate of the mean value of the peak, RPT_MEAN, and the 95% confidence bounds on the mean, [XMIN,XMAX].

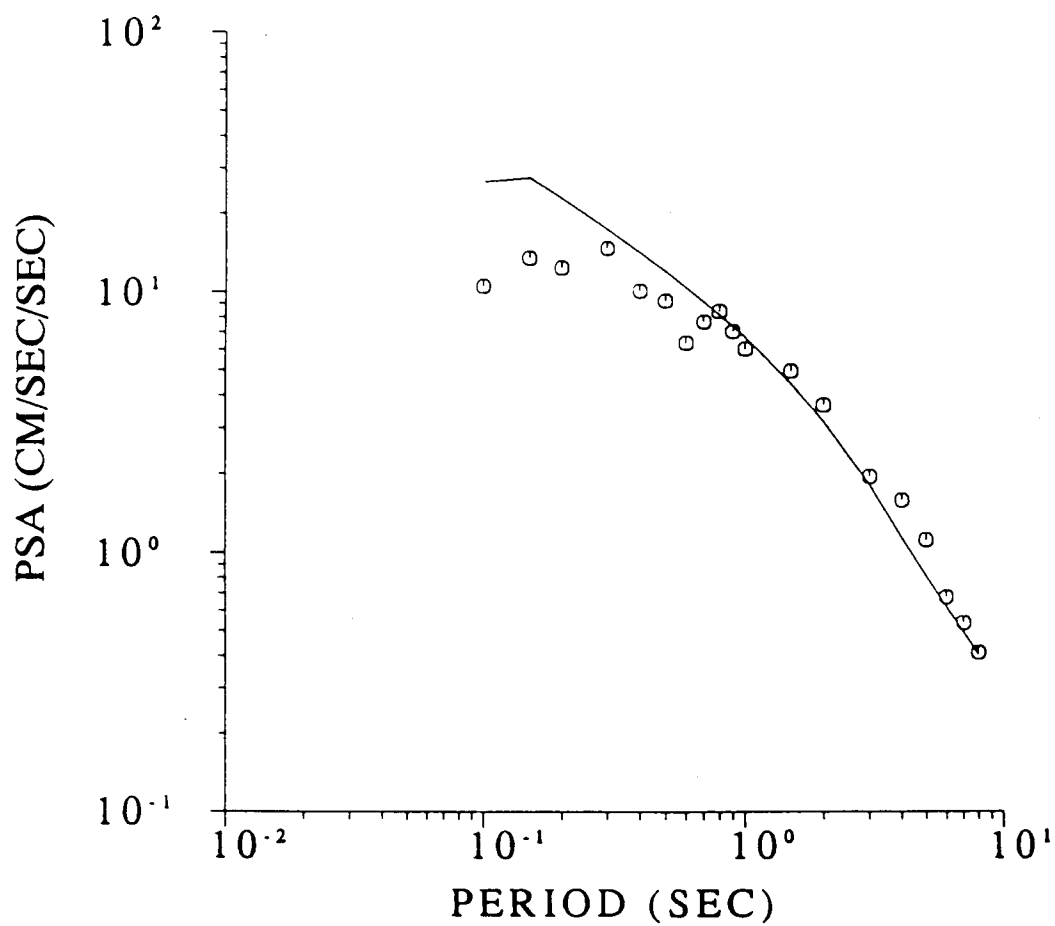


Figure 3.A-2

Comparison of RPT estimate of pseudo-spectral acceleration (PSA) from the design spectra (solid line) to the values from the time domain realization (circles). For this source time function, the RPT overestimates the time domain peak values at short periods. The wave propagation simulations will yield similar differences when the same source time function is used.

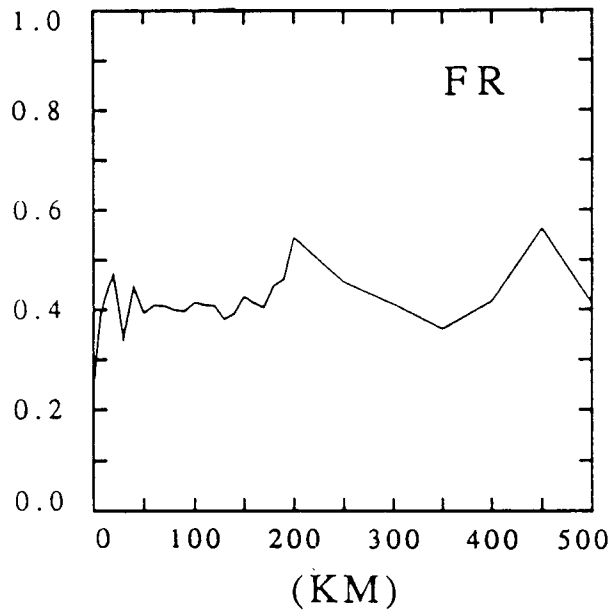
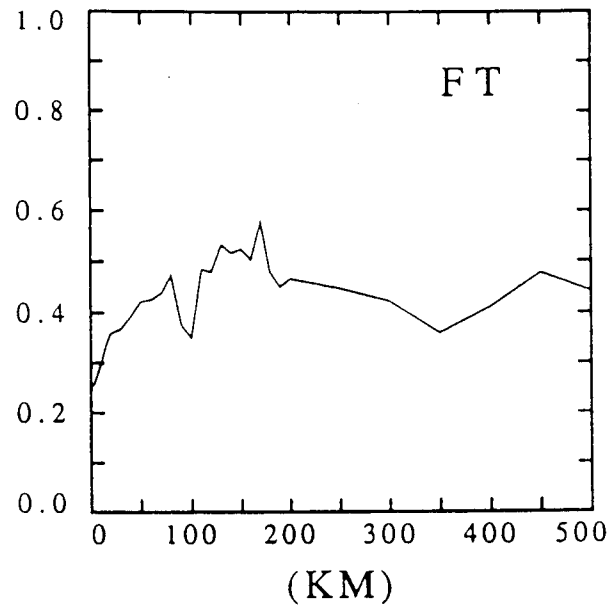


Figure 3.A-3
Source excitation factors, F , for the Mid-continent Model for a depth of 10 km as a function of epicentral distance.

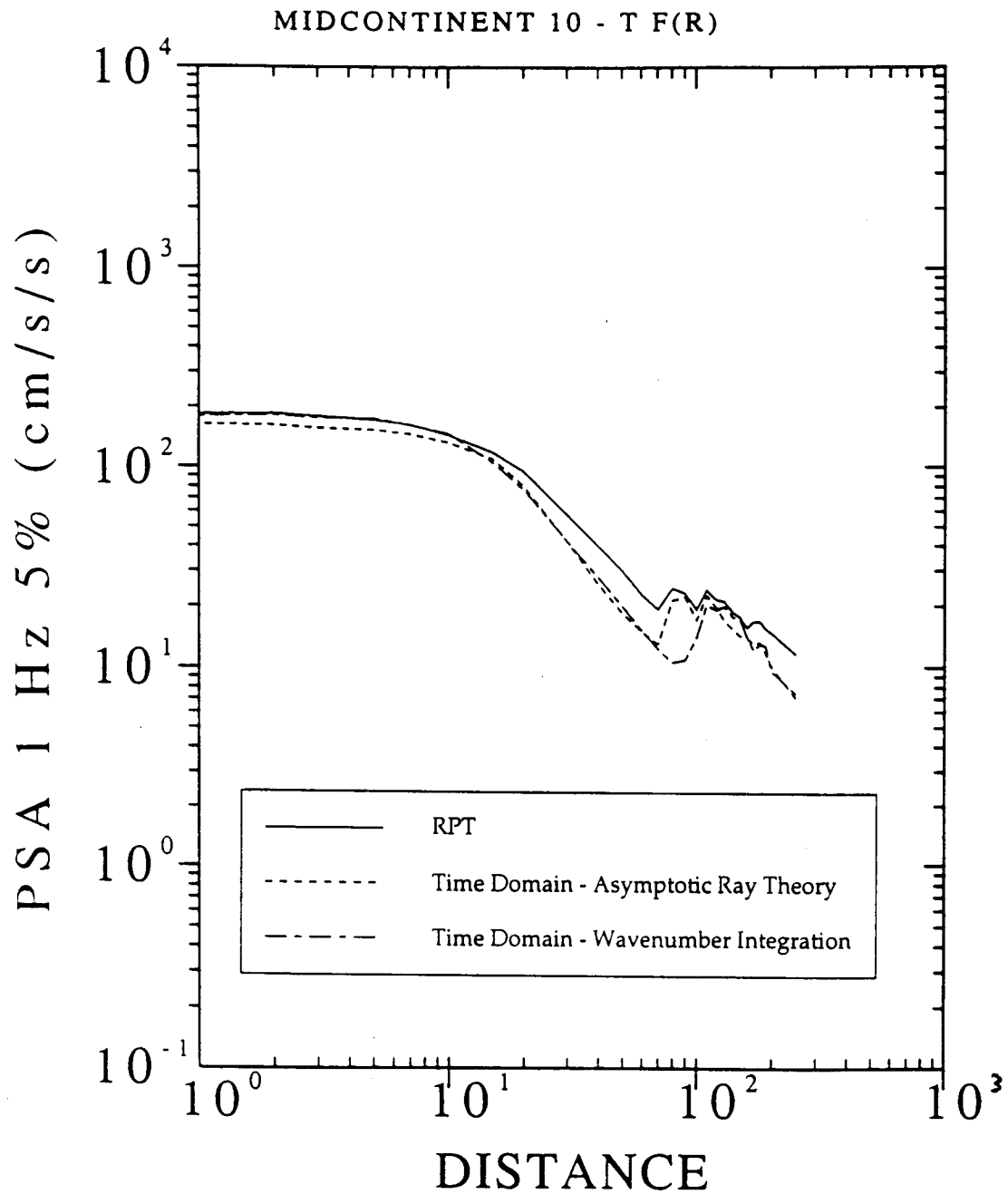


Figure 3.A-4a

Comparison of the RPT and time-domain estimates of the 1.0 Hz PSA, for the transverse component. The Mid-continent model and a focal depth of 10 km are assumed. A distance dependent F factor is used. The RPT values should be about 10% larger than the time domain methods due to the source time function (See Fig. 3.A-2).

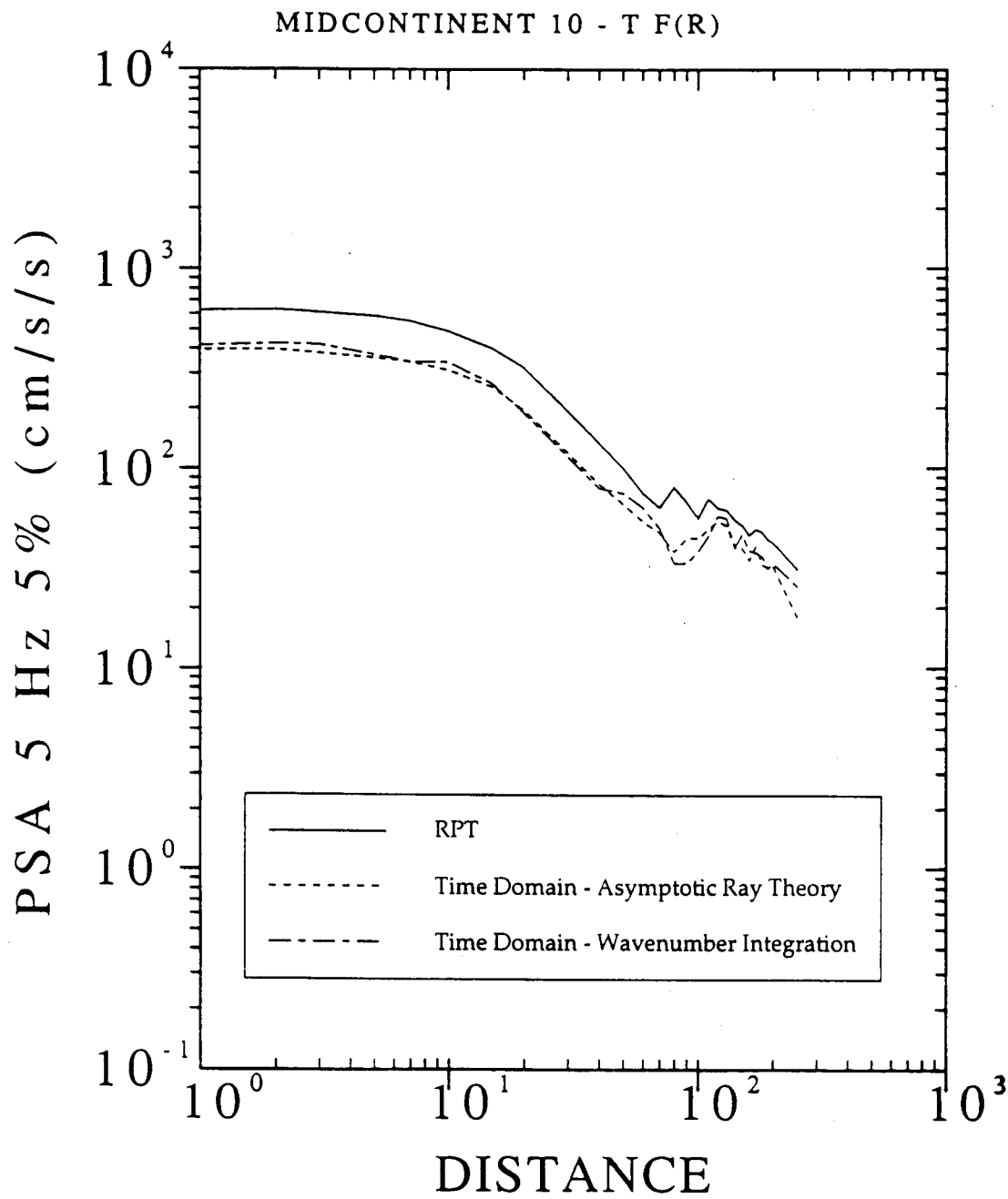


Figure 3.A-4b

Comparison of the RPT and time-domain estimates of the 5.0 Hz PSA, for the transverse component. The Mid-continent model and a focal depth of 10 km are assumed. A distance dependent F factor is used. The RPT values should be about 80% larger than the time domain methods due to the source time function (See Fig. 3.A-2).

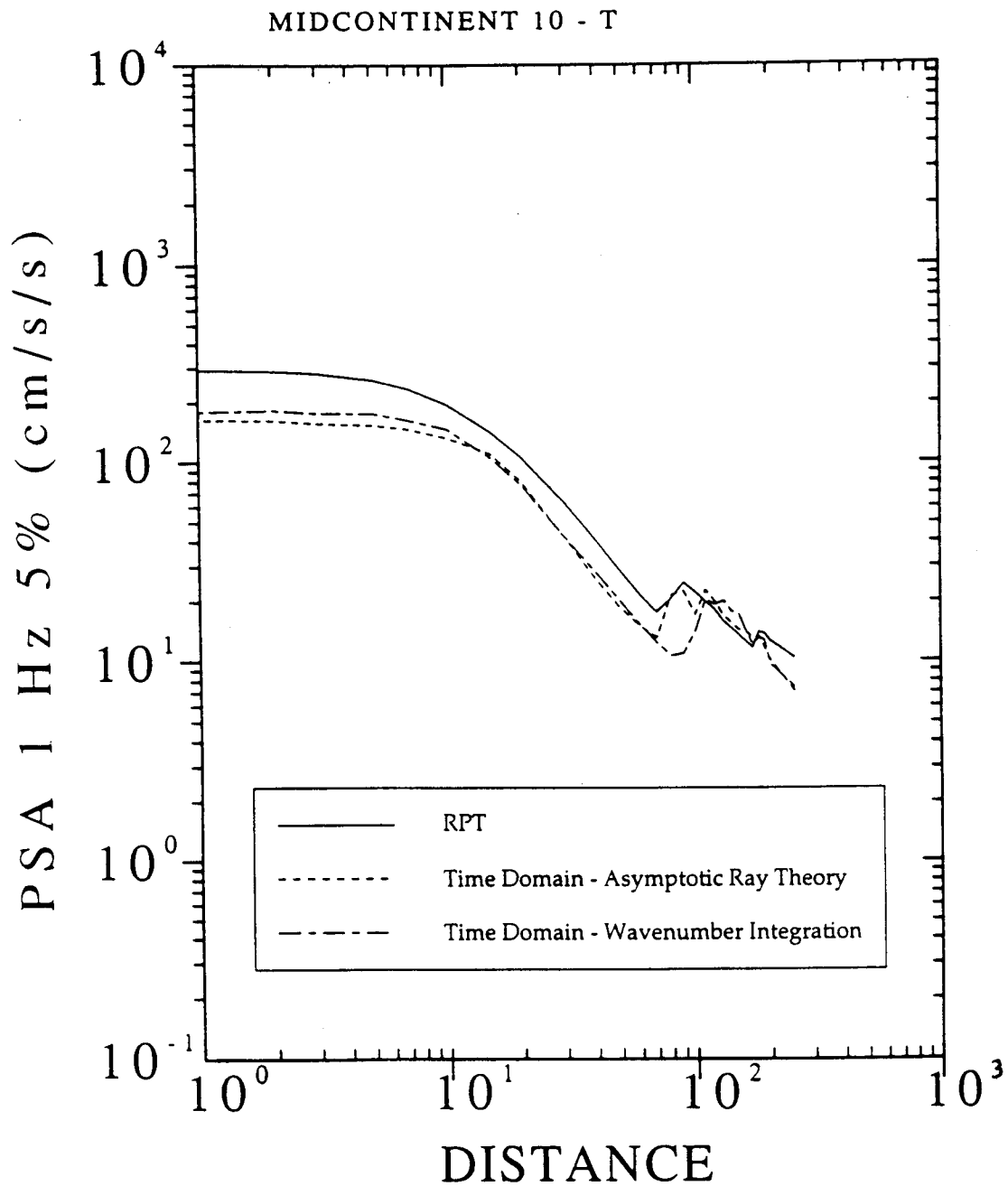


Figure 3.A-5a

Comparison of the RPT and time-domain estimates of the 1.0 Hz PSA, for the transverse component. The Mid-continent model and a focal depth of 10 km are assumed. A distance independent F factor of 0.42 is used which results in additional conservatism in the RPT estimate at short distances. The RPT values should be about 10% larger than the time domain methods due to the source time function (See Fig. 3.A-2).

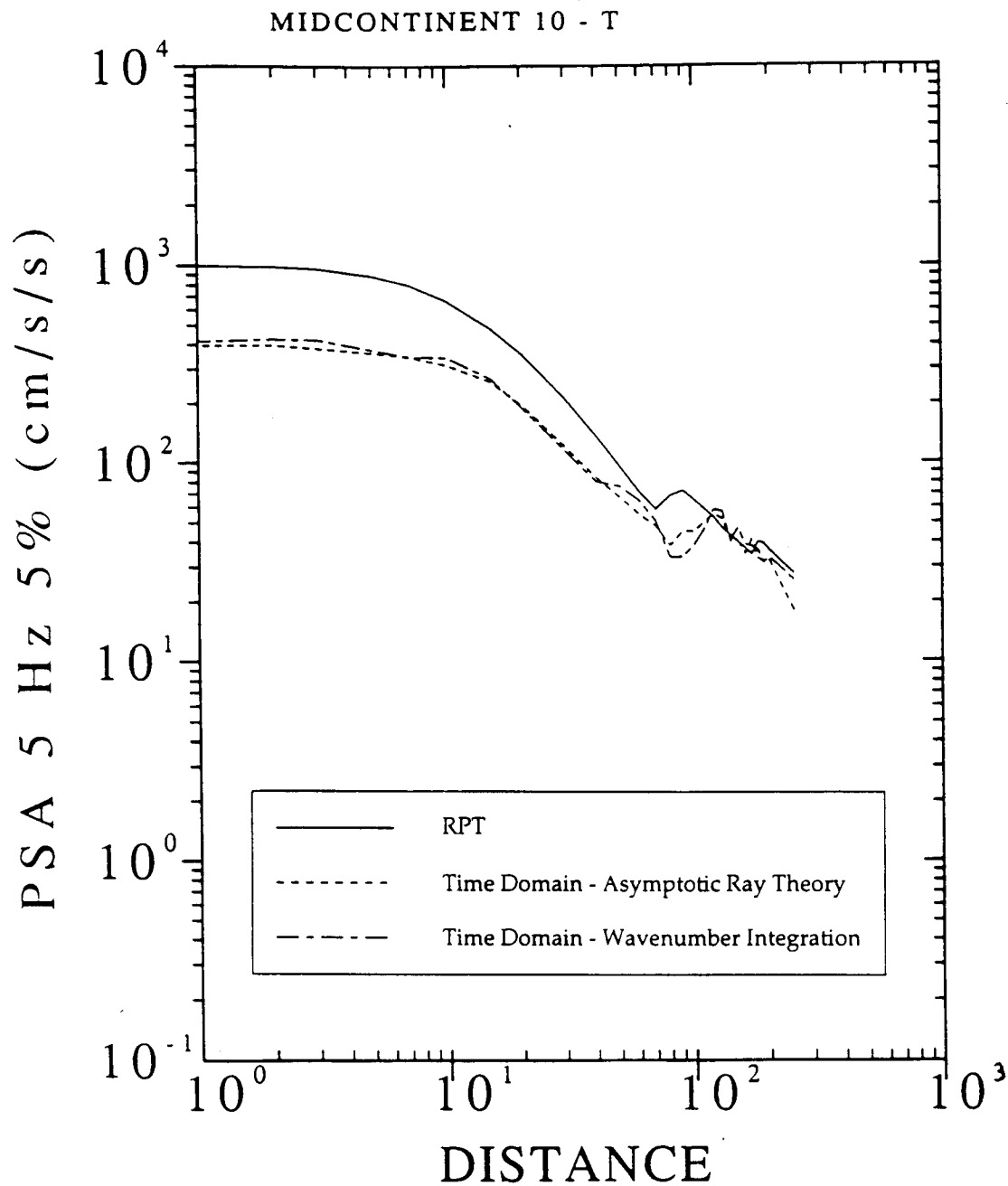


Figure 3.A-5b

Comparison of the RPT and time-domain estimates of the 5.0 Hz PSA, for the transverse component. The Mid-continent model and a focal depth of 10 km are assumed. A distance independent F factor of 0.42 is used which results in additional conservatism in the RPT estimate at short distances. The RPT values should be about 80% larger than the time domain methods due to the source time function (See Fig. 3.A-2).

APPENDIX 3.B

GENERALIZED RAY PROCEDURE

FOR MODELING GROUND MOTION ATTENUATION

This appendix describes details of the semi-empirical ground motion modeling method used to regionalize crustal structure for propagation path effects. A summary of the method is contained in Section 3.3. We describe here the summation procedure used to simulate the effects of fault rupture at the source and the use of empirical source functions to represent the high-frequency radiation and wave propagation, together with other details of the process needed to realistically reproduce observations of fault rupture.

3.B.1 Summation Procedure

The starting equation for the method is the representation theorem integral (Aki and Richards, 1980, eqn. 14.1) which gives the elastic displacement u caused by a displacement discontinuity $|u_j(\zeta, \tau)|$ across an internal surface Σ as

$$u_i(\mathbf{x}, t) = \int_{-\infty}^{\infty} d\tau \int_{\Sigma} |u_j(\zeta, \tau)| c_{ijpq} G_{ip,q}(\mathbf{x}, t; \zeta, \tau) v_k d\Sigma(\zeta) \quad (3.B-1)$$

where c_{ijpq} are the elastic constants; G_{ip} is the Green function and $\hat{v}$ is unit normal to the fault surface. The ζ represents a point on the fault plane and $\mathbf{x}$ is the observation point. In the far-field, the above integral can be written as

$$U(\mathbf{x}, t) = \int_0^L \int_0^W \dot{D}(\zeta, \eta, t) * G(\mathbf{x}, \zeta, t) d\zeta d\eta \quad (3.B-2)$$

where $\zeta = (\zeta, \eta)$ is the point on the fault plane, L is the fault length, W is the fault width, D is slip time history and G is the impulse response of the medium, and $*$ represents a convolution. When we use the above equation to calculate the ground motion due to a large fault, we divide the fault plane into small elements, called subfaults. Assuming that we break up the fault plane into l elements ΔL along the length and m elements (of width ΔW) along the width of the fault plane, the above integral reduces to the following form

$$U(\mathbf{x}, t) = \sum_{i=1}^l \sum_{j=1}^m \int_0^{\Delta L} \int_0^{\Delta W} \dot{D}(\zeta, \eta, t - \tau_r^{ij}) * G(\mathbf{x}, \zeta, t) d\zeta d\eta \quad (3.B-3)$$

where τ_r^{ij} is the time that the rupture front takes to propagate from the hypocenter to the center of the fault element (i, j) . The propagation time from the element (i, j) to the observation point is included in the Green's function.

Hadley and Helmberger (1980) and Irikura (1983) pointed out the need to take into account the difference between the dislocation rise time of the large event and that of the small event. Based on a Haskell type self-similar kinematic source model, we need to add as many sources per subfault as the ratio of the rise time of the large event to that of the subevent from which the source function is derived. Then the above equation reduces to the following form

$$U(x,t) = \sum_{i=1}^l \sum_{j=1}^m \sum_{k=1}^{nsrc} \int_0^{\Delta L} \int_0^{\Delta W} \dot{D}(\zeta, \eta, t - \tau_r^{ij} - (k-l)\tau_e) * G(x, \zeta, t) d\zeta d\eta \quad (3.B-4)$$

where $nsrc$ is the number of subsources per subfault given by $nsrc = \tau_o / \tau_e$, where τ_o and τ_e represent the rise time of the large event and the subevent, respectively. The simulated motion of the large event is then the summation of contributions from l fault elements along strike, m fault elements down dip, and $nsrc$ source functions lagged in time on each fault element. The rise time is estimated from an empirical relationship derived from data summarized by Heaton (1990).

3.B.2 Accommodation of High Frequency Radiation

3.B.2.1 Empirical Source Functions

Although the form of the radiated source spectrum may be specified deterministically, high frequency details are generally unknown and are therefore included empirically through the empirical source function. The empirical source function is derived from a strong motion recording close to a smaller event that has been corrected by a simplified Green's function that includes only geometrical spreading. The correction that is most consistent with the usage of the source functions is a deconvolution of the recording with a theoretical Green's function from the source function event to the receiver. Due to zeros in the spectrum, however, deconvolution is typically unstable. Since the recordings of the source functions are from distances of about one source depth, the synthetic response functions are quite simple (usually a step). In this case, the deconvolution amounts to division by a constant, G_o , which is the amplitude of the step function. G_o is estimated by comparing the maximum amplitude of the seismogram with its maximum after convolution with the step function. That is

$$G_o = \frac{\max(S(t) * G(t))}{\max(S(t))} \quad (3.B-5)$$

where $S(t)$ is the recorded seismogram, and $G(t)$ corresponds to the Green's function for the P, SV and SH waves from the source function event to the recording instrument. The empirical source functions also include an estimate of the frequency-dependent receiver functions. That is, they include the angular decomposition of the P, SV and SH wavefields onto vertical, radial and tangential components (PZ, PR, SVZ, SVR, SH), as well as amplification effects due to the medium properties at the surface. Since the velocity structure at the strong motion site for the large event may be different from the receiver structure of the empirical source functions, and since incidence angles can be quite different, a correction must be applied to the estimated receiver functions. In the simulation procedure, this correction is performed by multiplying the empirical source function by the ratio of theoretical whole-space receiver functions, producing an angle of incidence appropriate for signals arriving at the site.

The empirical source function contains several effects in addition to that of the source. These include anelastic absorption (Q), unmodeled wave propagation effects such as scattering, and local site response. At present, the only sets of appropriate empirical source functions available for use with this simulation method are from earthquakes in California. We have used suites of strong motion recordings of two small earthquakes to develop empirical source functions for use in simulating crustal earthquakes. These are the 23:19 aftershock of the October 15, 1979 Imperial Valley earthquake (Liu and Helmberger, 1985) and the October 4, 1987 aftershock of the 1987 Whittier Narrows earthquake (Yoshimura and Saikia, 1991).

To use these California source functions in the eastern United States, we modified the empirical source functions to give them a flat Fourier amplitude spectrum in acceleration at 10 Hz and above. This removes the effect of anelastic absorption from these source functions, which on average was measured to be a t^* of 0.55 for each event. The effect of anelastic absorption in simulations using these modified source functions is included computationally using the desired value of t^* .

3.B.2.2 Simplified Green's Functions

Simplified Green's functions for each fault element are computed using the method of generalized rays (Helmberger and Harkrider, 1978; Aki and Richards, 1980). The response is computed for a point source located at the center of the fault element with propagation through a plane-layered medium. Rays corresponding to the direct, upgoing P and S waves as well as primary reflections from each layer interface beneath the source are included. Mode conversions, reverberations and surface waves are not included. Since the radiation pattern and receiver function are implicitly included in the empirical source function, the synthetic Green's functions do not contain these factors. The Green's functions are thus essentially the response of the medium for P, SV and SH potentials. Three (P, SV and SH) step responses are computed for each fault element-site pair. The ray parameter of the first arriving ray (typically the upgoing direct ray) is used for the receiver function correction in the simulation procedure.

3.B.2.3 Empirical Representation of Frequency Dependence of Radiation Coherence

At low frequencies, nodes and lobes of the radiation pattern of an earthquake are readily observed. However, at high frequencies, radiation pattern nodes are generally not observed. In particular, the increasing incoherence in the radiation pattern in the strong motion recordings of the October 15, 1979 23:19 Imperial Valley aftershock has been noted (Liu and Helmberger, 1985; Wald et al., 1988). This is presumably due to deviations from a planar fault and scattering from small-scale heterogeneities which tend to randomize the radiation pattern at high frequencies. We have incorporated this frequency-dependent radiation pattern empirically. This was done by choosing from a suite of source functions based on their original location on the focal sphere. The empirical source functions used for a given fault element are those whose low-frequency, theoretical radiation pattern value are closest in absolute value to the theoretical radiation pattern value required for that fault element. To avoid the repetitive use of a single source function, a suite of source functions is selected whose theoretical radiation pattern values lie close to the required value.

3.B.2.4 Stochastic Components of Rupture Velocity and Slip Velocity

Each fault element begins to radiate energy when the rupture front reaches that element. To avoid artificial periodicities due to the discretization of the fault plane, and to approximate irregularities in rupture propagation, we have incorporated a stochastic component in the time at which each fault element begins to radiate. As illustrated in Figure 3.B-1a, the turn-on time, T_0 , includes a random number RG with a Gaussian probability distribution centered about $(t_b - t_a)/2$. The probability that T_0 is within (t_a, t_b) was set at the three standard deviation level, or a confidence level of 99.7%.

The form of the slip function at each point on the fault is assumed to be a simple ramp (following Haskell, 1969), parameterized by static slip and rise time. The slip function of the large event is built up by the summation of nsrc empirical source functions as described above. In order to avoid an artificial periodicity corresponding to the rise time of the source function, and to simulate roughness or variation in the instantaneous slip rate, a stochastic component is included in the slip function. Accordingly, in addition to lagging the source functions at uniform multiples of the rise time T_e of the source function, a random lag is also included as illustrated in Figure 3.B-1b. The probability that the initiation time t_{si} of a given source function is within (t_i, t_i+1) was set at the three standard deviation level.

3.B.2.5 Heterogeneous Slip Distribution (Asperities)

An important aspect of the source at high frequencies is the characterization of fault heterogeneity or roughness. The heterogeneity may be manifested as local variations in static slip, slip rate, or rupture velocity. It is possible that all three are interdependent and that each contributes significantly to the high frequency radiation. There is little agreement, however, in exactly how these characteristics are physically related, and it is unreasonable to allow them to vary independently. We have therefore opted to implement a simple asperity model of fault roughness in which static slip varies in a deterministic manner over the rupture surface, while variations in slip rate and rupture velocity are incorporated stochastically. There is now a large set of earthquakes for which the distribution of slip over the rupture surface has been estimated from the inversion of strong motion and teleseismic records. To specify the slip distribution on the fault, each element is assigned a weighting factor which is a discretization of the static slip contours, normalized to preserve total seismic moment.

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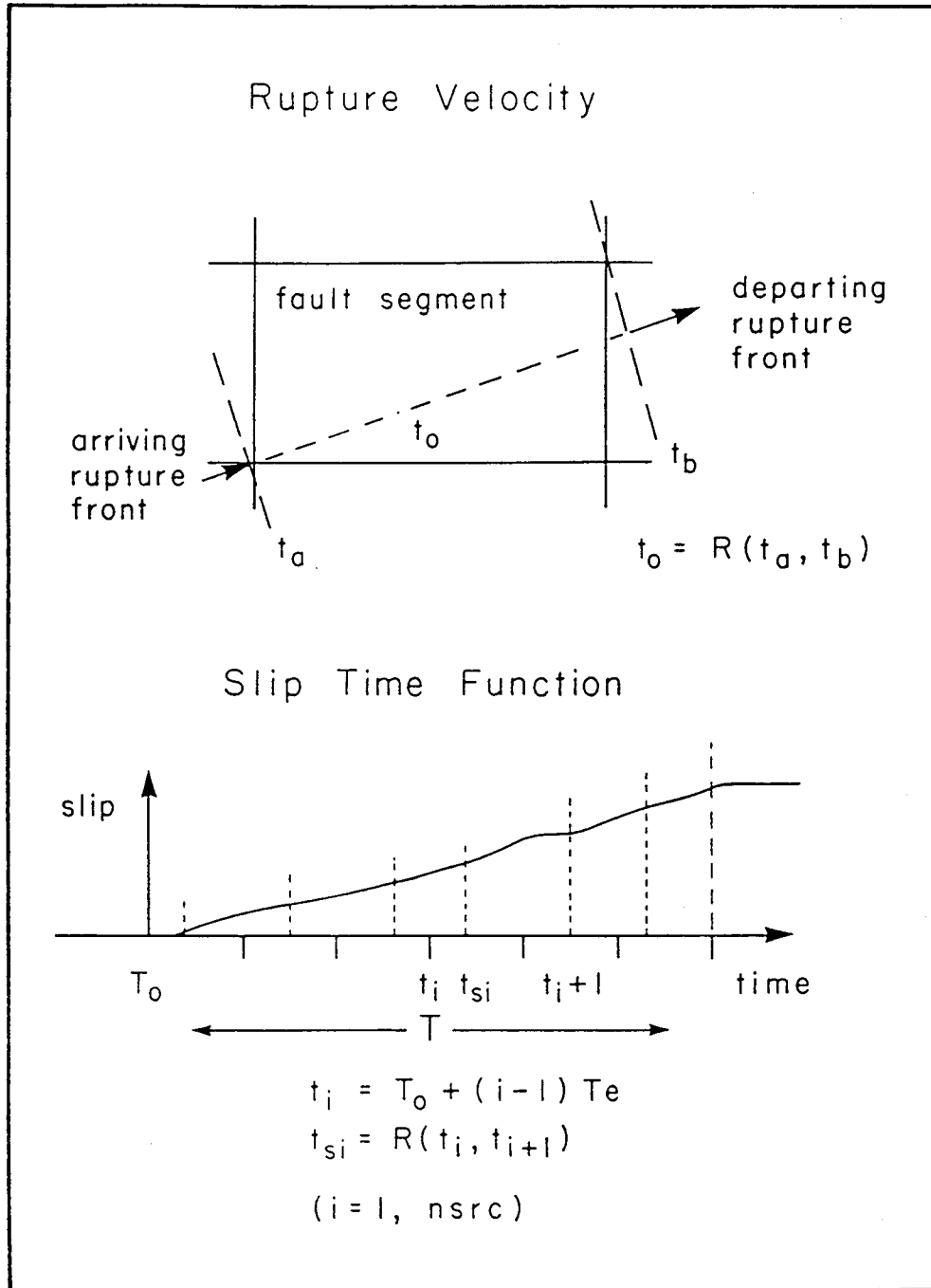


Figure 3.B-1

Schematic diagram illustrating random component of (a) rupture velocity and (b) slip velocity. The rupture onset time T_0 for each fault element is a random number having a Gaussian distribution centered between the arrival and departure times of the rupture front. The initiation time t_{si} for each of the $nsrc$ source functions is a random number having a Gaussian distribution centered on evenly spaced intervals of the source function's rise time Te . Source: Somerville et al., 1990.

APPENDIX 5.A

CRUSTAL MODELS FOR VELOCITY REGIONALIZATION

The sixteen crustal structure regions that are summarized in Section 5.1 are described in detail in this appendix. The crustal models for the individual regions are listed in Table 5.A-1 and are shown in Figures 5.A-1 through 5.A-16.

5.A.1 Offshore New England

This crustal province includes southern New England and the offshore area adjacent to New England. Taylor (1989) compiled three crustal models for this region, which spatially corresponds with the Avalon terrane. Based on these models, we characterize the province using a one-layer model, even though one of those presented by Taylor (1989) suggests a two-layer model. The upper layer, with a thickness of 30 km and a P-wave velocity of 6.5 km/sec, overlies mantle material with a velocity of 8.1 km/sec (Taylor, 1989).

5.A.2 Northern Appalachians

This province consists of the central mobile belt of the Northern Appalachians. These rocks probably represent island arc material that was accreted to the North American craton during the Taconic orogeny (middle Ordovician), and later deformed during the Acadian orogeny (Devonian) (e.g., Taylor, 1989). It includes the Merrimack Synclinorium and the Bronson Hill Anticlinorium and extends from southern New England north into southeastern Canada.

Taylor (1989) compiled four crustal models that were developed for this area. Three of them characterize the crust by three layers; one uses four layers. The models agree that the upper crustal velocity is 6.0 to 6.1 km/sec and the lower crust has a velocity of about 7.0 to 7.2 km/sec. The velocity of the intermediate crustal layer in the three-layer model is about 6.8 km/sec. To the north, Braile (1989) interprets a Northern Appalachian model with two layers: 6.35 km/sec in the upper crust and 7.35 km/sec in the lower crust. Taylor (1989) suggests that the higher velocity lower crust is related to oceanic material that either underlay an accreted island arc or was associated with a former fore-arc or back-arc basin.

Based on available information, we model the crust in this province using two layers: an upper layer with a P-wave velocity of about 6.0 km/sec and a thickness of about 20 km over a lower crustal layer with a velocity of about 7.1 km/sec and a thickness of 20 km. Underlying these layers the upper mantle has a velocity of about 8.1 km/sec.

5.A.3 Atlantic Coastal Plain

The Atlantic Coastal Plain province extends from New Jersey down to Georgia and extends inland to the Inner Piedmont. The crust here is thinner relative to that found beneath the central Appalachians and the Valley and Ridge to the west. The boundary between the Atlantic Coastal Province and the Offshore New England Province is uncertain.

In this province sedimentary sequences overlie the crystalline basement. Towards the west, the two layers are likely separated by a decollement surface. The eastern extent of the decollement is not well defined (e.g. Taylor, 1989). The basement rock here is interpreted to have an African or European affinity. Taylor (1989) compiled four crustal velocity models from within this province. These models characterize the crust using either two or three layers. In the three layer models, a thin sedimentary sequence is underlain by intermediate crust with a velocity of about 6.0 km/sec. Lower crust in these models is thin and

characterized by a velocity of about 6.7 km/sec. The two layer models consist of an upper crust of about 6.0 km/sec and a lower crust of about 6.5 km/sec.

Based on the crustal models and tectonic interpretation presented in Taylor (1989) and Braile (1989), we model the Atlantic Coastal province using two layers. The upper layer has a P-wave velocity of about 6.0 km/sec and a thickness of 18 km, overlying a lower crust with a velocity of about 6.5 km/sec and a thickness of 18 km (Talwani, 1982; Taylor, 1989). Upper mantle velocities are about 8.1 km/sec.

5.A.4 Gulf Coastal Plain

The Gulf Coastal Plain province extends along the southern border of the study area from Florida through Texas. It is bounded to the north by the Ouachita trend. In addition to the Ouachita trend, we also use the 30 km contour of crustal thickness (Braile, 1989) to guide selection of the northern boundary for this province. To the east, its boundary with the Atlantic Coastal province is uncertain.

Worrall and Snelson (1989) and Trehu and others (1989) summarize the structure and evolution of the Gulf coastal margin. The coastal plain is underlain by crust that is transitional between the Ouachita basement and oceanic crust of the Gulf of Mexico. Beneath the coastal plain, crustal models indicate low velocities typical of sedimentary deposits to a depth of about 15 km, below which occurs basement with a velocity of about 6.5 km/sec. Beneath the continental shelf offshore, a lower crustal layer with velocity of about 7.3 is observed. In the northern part of this province, data are also available from a PASSCAL profile across the Ouachita front (Keller and others, 1989).

In this province we model the structure using three layers. The top layer is characterized by a P-wave velocity of about 4.0 km/sec and a thickness of 7 km; the middle layer exhibits velocities of about 5.3 km/sec and a thickness of 8 km; the lower crustal layer has velocities of about 6.5 km/sec and a thickness of 15 km (Worrall and Snelson, 1989). Upper mantle velocities are about 8.2 km/sec.

5.A.5 Southern Appalachians

The Southern Appalachians province covers the region extending from northern Georgia to southern Pennsylvania in which the crust thickens to about 45 km. This province corresponds approximately with the Blue Ridge and the Valley and Ridge geologic provinces of the Appalachians. These geologic provinces are underlain by Grenville basement rocks.

Taylor (1989) compiled five crustal models determined for this province. The models indicate an upper crustal layer with velocity of about 5.7 to 6.2 km/sec underlain by a layer having a velocity of about 6.5 to 6.7 km/sec. The thickness of the upper layer ranges from 10 to 22 km in the different interpretations. Braile (1989) based on one of the same data sets compiled by Taylor (1989) interprets the crust in this region using three layers having velocities of 6.01, 6.33 and 6.73 km/sec, respectively.

Based on the available refraction data, and on the interpretations of Braile (1989) and Taylor (1989), we choose to model the crust in this province using three layers. The upper layer has a P-wave velocity of about 5.9 km/sec and a thickness of 5 km. The intermediate layer has a velocity of about 6.2 km/sec and a thickness of 10 km. The lower crust has a velocity of about 6.7 km/sec and a thickness of about 30 km. The upper mantle velocity is about 8.1 km/sec.

5.A.6 Central Tennessee

The Central Tennessee province lies along the eastern edge of the Appalachian plateau centered in Tennessee. It is underlain by basement of the Eastern Granite-Rhyolite age province (Bickford and others, 1986), and is characterized by a relatively thick crust that includes a high-velocity layer at its base (Braile, 1989). The northern and southern extent of this province are delineated on the basis of a decrease in crustal thickness.

The Central Tennessee province is differentiated from the Western Tennessee province based on the work of Prodehl and others (1984) as summarized by Braile (1989). The velocity and thickness of the upper crust appear to be significantly different in central and western Tennessee.

Following Braile (1989), we define a three layer model for this region. The upper crust has a P-wave velocity of 6.4 km/sec and thickness of 20 km; the intermediate layer has a velocity of 6.8 km/sec and a thickness of 15 km; and the lower crust has a velocity of 7.3 km/sec and a thickness of 15 km. The upper mantle velocity is about 7.9 km/sec.

5.A.7 Western Tennessee

The Western Tennessee province encompasses the region between the Central Tennessee province to the east and the New Madrid rift complex to the west. It lies along the southwestern edge of the Eastern Granite-Rhyolite age province (Bickford and others, 1986). The crust here is relatively thick and includes a lower crustal layer with high velocity. In comparison to the Central Tennessee province to the east, it has a thinner upper crustal layer with a lower velocity (Braile, 1989). The northern and southern boundaries are based on changes in crustal thickness.

Following Braile (1989), we interpret the crust in this province to consist of three layers. At the top of the crust, a layer with a P-wave velocity of about 6.1 km/sec extends to a depth of 10 km. This layer is underlain by one with a velocity of 6.7 km/sec and a thickness of 30 km. The lower crust has a velocity of about 7.4 km/sec and thickness of 10 km. The upper mantle velocity is about 8.0 km/sec.

5.A.8 New Madrid Rift

Crustal structure in the New Madrid Rift province has been extensively studied because of its history of major earthquake activity. The province is defined to include the Reelfoot rift, the Southern Indiana (Wabash) rift arm, the St. Louis rift arm and the Rough Creek Graben. Most recent studies of crustal structure have concentrated on the Reelfoot rift (e.g., Ginszburg and others, 1983; Mooney and others, 1983), which is likely associated with the sequence of the large earthquakes in the winter of 1811–1812. McCamy and Meyer (1966) analyzed refraction data along a northeast-southwest profile just outside the Reelfoot rift to the northwest.

Detailed analyses of refraction data from the northern Mississippi embayment, including ray tracing and the use of synthetic seismograms, yielded a two-dimensional model of the velocity structure (Ginszburg and others, 1983). In this study, we simplify these results and present a one-dimensional model consisting of four homogeneous layers. The upper layer extends to 3 km and has a P-wave velocity of about 5.9 km/sec. This is underlain by a layer with a thickness of 10 km and a velocity of about 6.2 km/sec. The third layer has a velocity of about 6.6 km/sec and a thickness of 12 km. The lower crust has a velocity of about 7.3 km/sec and a thickness of 20 km.

Thus, the Reelfoot rift is characterized by a thinned upper crust and a thick lower-crustal layer of high-velocity. The high-velocity lower crust is consistent with the interpretation of the structure as a Precambrian rift (e.g. Braile and others, 1982). The upper mantle in this province exhibits a P-wave velocity of about 8.0 km/sec.

5.A.9 Ozarks

The Ozarks province is centered on the border between Missouri and Arkansas. It is underlain by basement belonging to the Eastern Granite-Rhyolite age province (Bickford and others, 1986). It is bounded to the south by the Ouachita trend and to the north by the Central Plains age domain of Bickford and others (1986). Refraction data for northern Missouri (Stewart, 1968) indicate that the velocity structure in that region differs from the one in southern Missouri.

Stewart (1968) analyzed refraction data from the Ozarks province and characterized the crustal structure with a three layer model. In contrast with interpretations for data to the east and west (see Braile, 1989,

Figure 5.1.1-3), Stewart's model has an anomalously low velocity for the intermediate layer (6.1 versus about 6.5 km/sec). Stewart (1968) notes that the data for the southern Missouri profile were poor and difficult to interpret consistently. Thus, the velocity structure he derived should be viewed as uncertain.

Taking Stewart's (1968) results at face value, we model the crust of the Ozarks province using three layers. The model consists of an upper layer with a velocity of about 6.0 km/sec and a thickness of 10 km, an intermediate layer with a velocity of about 6.1 km/sec and a thickness of 20 km, and a lower crustal layer with a velocity of about 7.3 km/sec and a thickness of 12 km. The upper mantle velocity is about 8.0 km/sec.

5.A.10 Northern Grenville-Superior

The Northern Grenville-Superior province covers the Grenville terrane from Ohio and Pennsylvania north into Canada, and also the Superior terrane except in the vicinity of Lake Superior. These regions consist of Precambrian basement that was relatively underformed during Phanerozoic orogenies.

Taylor (1989) compiled three crustal model interpretations for the Grenville province in New York. These models suggest a fairly uniform crust with a P-wave velocity of 6.4 to 6.6 km/sec; one model includes an upper crustal layer with a velocity of about 6.1 km/sec. Findings by Woodward-Clyde Consultants (unpublished data) for New York are consistent with the models summarized by Taylor (1989). In Canada (Braile, 1989; Taylor, 1989) and in eastern Ohio (Nicholson and others, 1988), data have been interpreted to suggest a two layer model.

Based on the results discussed above, we characterize the crustal structure of this province by a two layer crust. The upper layer has a P-wave velocity of about 6.2 km/sec and a thickness of 15 km; the lower layer has a velocity of about 6.8 km/sec and a thickness of 25 km. The upper mantle velocity is about 8.1 km/sec.

5.A.11 Lake Superior Basin

The Lake Superior Basin province centers on a region of thicker crust associated with a portion of the Midcontinent rift structure beneath Lake Superior and the border between Minnesota and Wisconsin. The Midcontinent rift structure extends in a north-northeasterly direction from Iowa to Lake Superior where it changes to a southeasterly trend and continues to southern Michigan. The area of thickened crust does not follow the entire rift structure; it is missing along the lower Michigan section.

Braile (1989) summarizes the results of several experiments using different approaches to understand the structure of the crust beneath the Midcontinent rift structure. The available data include both refraction and reflection profiles. The interpreted structure includes a basalt filled rift basin extending to about 20 km overlain by sedimentary deposits and underlain by a typical lower crustal layer. In this region the usual upper crustal layer is absent.

We model the crust in this province using two layers. The upper layer, with a P-wave velocity of about 5.2 km/sec and thickness of 5 km, represents sedimentary deposits that filled a subsiding rift basin. The lower layer, with a velocity of about 6.8 km/sec and a thickness of 40 km, consists of mafic material extruded into the rift basin and underlying typical lower basement of similar velocity. The upper mantle in this region is characterized by a velocity of about 8.1 km/sec.

5.A.12 Midcontinent

The Midcontinent province includes the Penokean age terrane and the northern portion of the Eastern Granite-Rhyolite age terrane (Bickford and others, 1986). It is differentiated from crustal provinces to the north and south by a reduced crustal thickness.

Braile's (1986) summary of crustal structure in the continental interior shows that while some data exist for the Penokean terrane, almost no data exist for the northern portion of the Eastern Granite-Rhyolite

terrane. This result in uncertainty concerning our decision to combine the Penokean terrane with the northern portion of the Eastern Granite-Rhyolite terrane. Of particular interest is whether the high velocity lower crust that characterizes the Eastern Granite-Rhyolite terrane to the south is also found in the northern portion of that terrane. Refraction results for the Penokean terrane do not suggest a high-velocity lower crust.

We model the Midcontinent province using a two layer crust that does not include a high-velocity lower layer. The model is based on results of Steinhart and Meyer as presented in Braile (1989). The upper crust exhibits a P-wave velocity of about 6.1 km/sec and a thickness of 12 km. It is underlain by a 28 km thick layer that is characterized by a velocity of 6.5 km/sec. The velocity of the upper mantle is about 8.0 km/sec.

5.A.13 Northern Great Plains

The Northern Great Plains province corresponds spatially with the Trans-Hudson orogen, a Precambrian terrane that separates the Superior and Wyoming cratons (e.g., Bickford and others, 1986). It also lies along the eastern edge of the Williston basin.

As interpreted by Braile (1989), this province represents a transition zone. To the west, the crust exhibits a relatively thick layer of high velocity material in the lower crust. In the Northern Great Plains province this layer is still observed, but is much thinner. At the same time, relative to the west, the upper and intermediate crustal layers thicken.

We describe the crust in this province using a three layer model. In the upper layer, which has a thickness of 10 km, the P-wave velocity is about 6.2 km/sec. The intermediate layer has a velocity of about 6.6 km/sec and a thickness of 25 km. The lower crust is characterized by a velocity of 7.1 km/sec and a crustal thickness of 15 km. The upper mantle velocity is about 8.1 km/sec.

5.A.14 Central Plains

The Central Plains province is differentiated from the Northern and Southern Great Plains provinces by its lack of a high velocity lower crust. The boundaries of this province are drawn to coincide with the Central Plains terrane of Bickford and others (1986). The crustal thickness of this province varies from about 40 km in the east to about 50 km in the west. For this regionalization we use a value of 45 km.

Prodehl and Lipman (1989) provide a summary of crustal structure results for the Rocky Mountain region. In the area of the western Central Plains province, they indicate a three layer model over a relatively low velocity upper mantle. In the eastern part of this province, Stewart (1968) also shows a three layer model for northern Missouri.

Based on the available data, we characterize the crust in this province using three layers. The upper crust has a P-wave velocity of about 4.5 km/sec and a thickness of 3 km; the intermediate layer has a velocity of about 6.2 km/sec and a thickness of 22 km; the lower crust has a velocity of about 6.7 km/sec and a thickness of 15 km. The upper mantle is characterized by a velocity of about 7.9 km/sec.

5.A.15 Southern Great Plains

The Southern Great Plains province is underlain by basement belonging to the Western Granite-Rhyolite age province (Bickford and others, 1986). It extends from Arkansas to New Mexico and from Nebraska to Texas.

The interpretive cross-section constructed by Braile (1989) along latitude 37° N passes through the middle of this province. Refraction results from the western and eastern parts of the Southern Great Plains province are similar, and they all include a high-velocity lower crust. In the central portion of the province, however, data are lacking.

To model the crust in this province, we use a four layer crust. The upper layer, consisting of sedimentary deposits, has a P-wave velocity of about 5.0 km/sec and a thickness of 2 km; the second layer exhibits a velocity of about 6.1 km/sec and a thickness of 14 km; the third layer has a velocity of about 6.7 km/sec and a thickness of 15 km; the lower crust is characterized by a velocity of about 7.2 km/sec and a thickness of 14 km. The velocity of the upper mantle is about 8.2 km/sec. The total crustal thickness is 45 km.

5.A.16 Williston Basin

The Williston Basin province is associated with a region of relatively thick crust centered on east-central Montana. In this region, total crustal thickness approaches 50 km. The basin is located on the Churchill terrane to the west of the Trans-Hudson orogen.

A number of refraction lines cover this region as summarized in Braile (1989) and Prodehl and Lipman (1989). Results consistently indicate the presence of high velocities in the lower crust. In the western portion of this province, the high velocity layer is relatively thick.

We represent the crustal structure of this region using a three layer model. The upper layer has a P-wave velocity of about 6.0 km/sec and a thickness of 10 km; the intermediate layer has a velocity of about 6.5 km/sec and a thickness of 15 km; the lower crust has a velocity of about 7.3 km/sec and is 25 km thick. The upper mantle velocity is about 8.2 km/sec.

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Table 5.A-1
Crustal Models

Crustal model for region 1 : Offshore New England

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	1	0
6.5	3.75	2.78	29	1
8.1	4.68	3.38	—	30

Crustal model for region 2 : Northern Appalachians

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	1	0
6.0	3.46	2.70	19	1
7.1	4.10	2.87	20	20
8.1	4.68	3.38	—	40

Crustal model for region 3 : Atlantic Coastal Plain

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	1	0
6.1	3.52	2.92	14	1
6.5	3.78	3.00	21	15
8.18	4.78	3.34	—	36

Crustal model for region 4 : Gulf Coastal Plain

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.0	2.31	2.37	7	0
5.3	3.05	2.58	8	7
6.5	3.76	2.78	15	15
8.2	4.74	3.40	—	30

Crustal model for region 5 : Southern Appalachians

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	1	0
5.63	3.44	2.64	5	1
6.05	3.52	2.70	9	6
6.53	3.84	2.78	35	15
8.18	4.7	3.33	—	50

Crustal model for region 6 : Central Tennessee

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	1	0
6.4	3.70	2.76	19	1
6.8	3.93	2.82	15	20
7.3	4.21	2.91	15	35
7.9	4.56	3.30	—	50

Crustal model for region 7 : Western Tennessee

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.53	1.5	0
6.1	3.53	2.71	8.5	1.5
6.7	3.87	2.81	30	10
7.4	4.28	2.92	10	40
8.0	4.62	3.35	—	50

Crustal model for region 8 : New Madrid Rift

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.7	2.72	2.48	4	0
6.2	3.58	2.73	5	4
6.5	3.76	2.78	21	9
7.4	4.28	2.92	15	30
8.1	4.68	3.35	—	45

Crustal model for region 9 : Ozarks

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	1	0
6.0	3.46	2.70	9	1
6.1	3.52	2.71	20	10
7.3	4.21	2.91	12	30
8.1	4.65	3.38	—	42

Crustal model for region 10 : Northern Grenville-Superior

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
5.0	2.34	2.53	1	0
6.3	3.64	2.74	3	1
6.4	3.70	2.76	2	4
6.5	3.76	2.78	2	6
6.6	3.82	2.79	12	8
6.7	3.87	2.81	10	20
6.8	3.93	2.83	8	30
8.1	4.68	3.35	—	38

Crustal model for region 11 : Lake Superior Basin

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	5	0
6.8	3.93	2.82	40	5
8.1	4.68	3.38	—	45

Crustal model for region 12 : Midcontinent

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	1	0
6.1	3.52	2.71	11	1
6.5	3.75	2.78	28	12
8.0	4.62	3.35	—	40

Crustal model for region 13 : Northern Great Plains

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	1	0
6.2	3.58	2.73	9	1
6.6	3.81	2.79	25	10
7.1	4.10	2.87	15	35
8.1	4.68	3.38	—	50

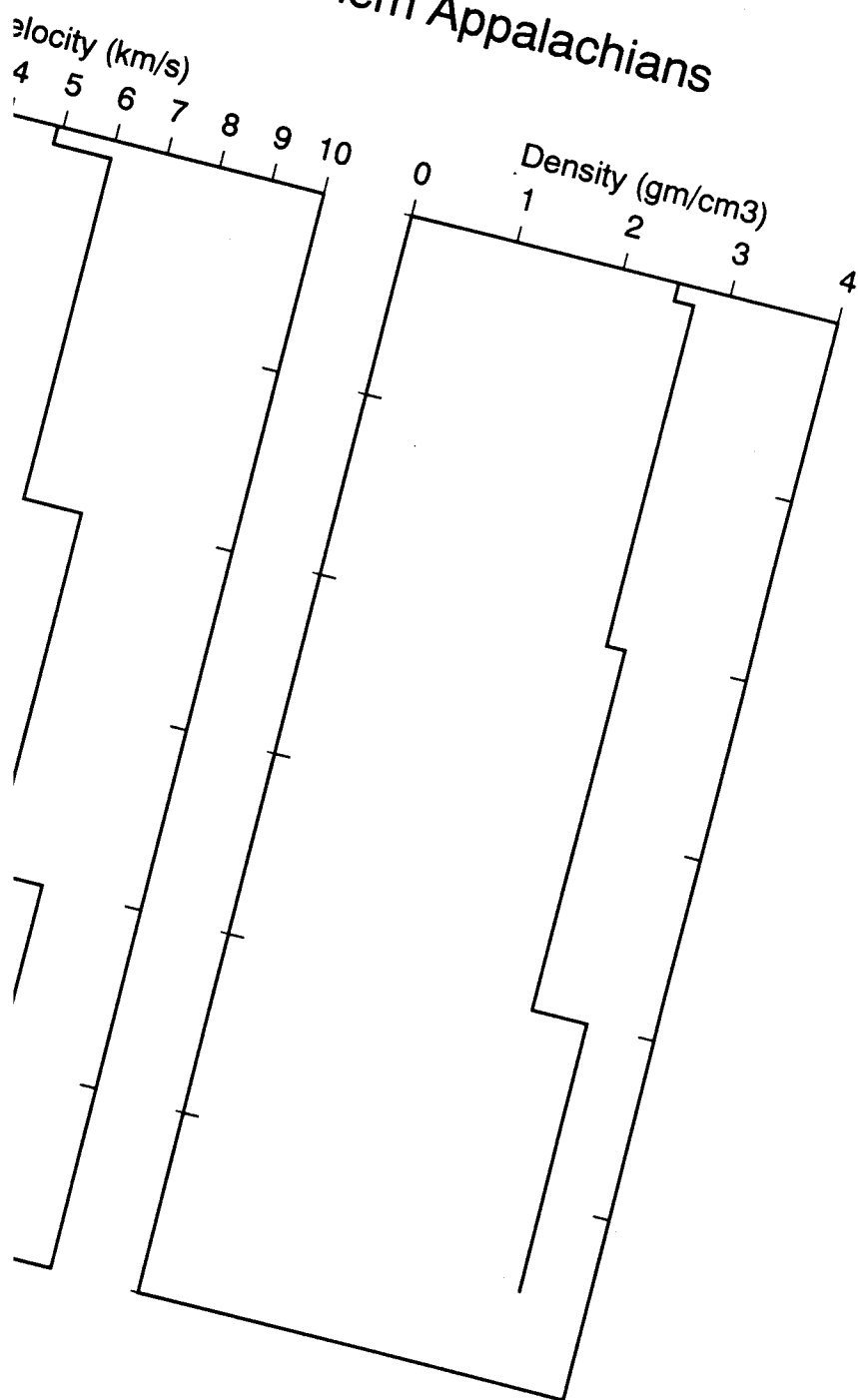
Crustal model for region 14 : Central Plains

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.5	2.60	2.45	3	0
6.2	3.58	2.73	22	3
6.7	3.87	2.81	15	25
8.0	4.62	3.35	—	40

thern Great Plains

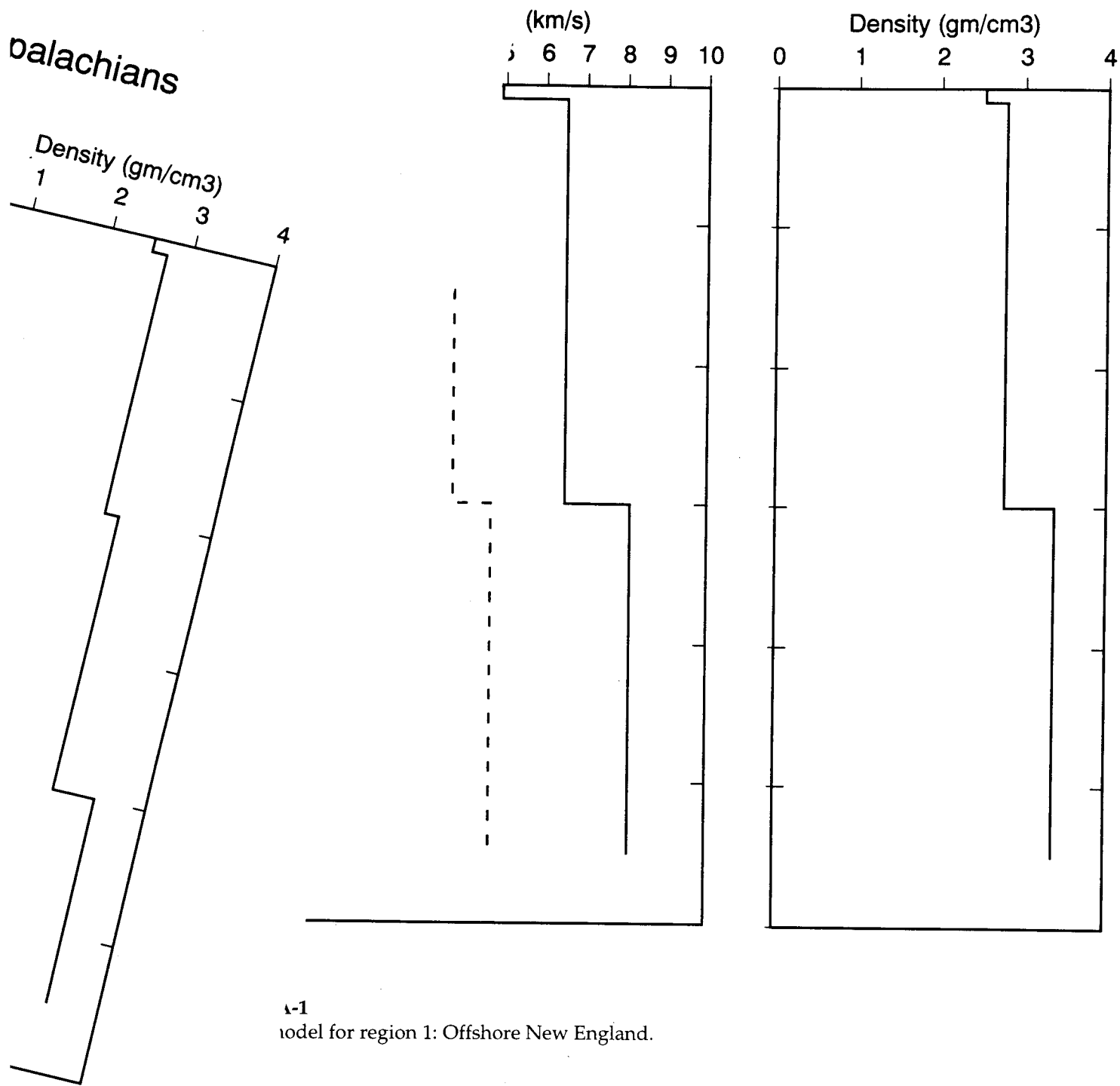
Depth to Top
 0
 2
 6
 30
 48

Region 2: Northern Appalachians



Depth to Top
 0
 1
 10
 25
 50

n 1: Offshore New England



Region 2: Northern Appalachians

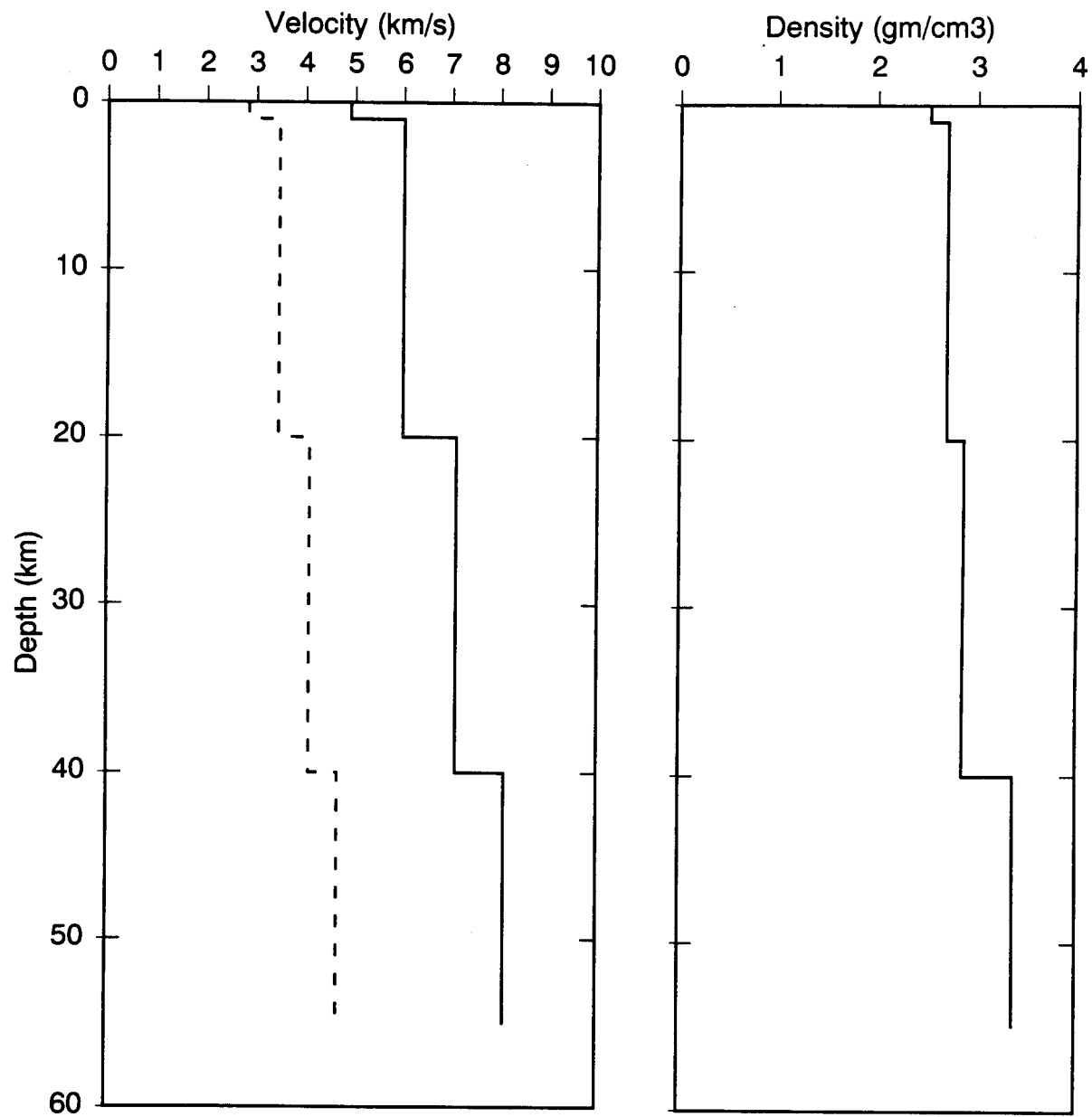


Figure 5.A-2
Crustal model for region 2: Northern Appalachians.

Region 3: Atlantic Coastal Plain

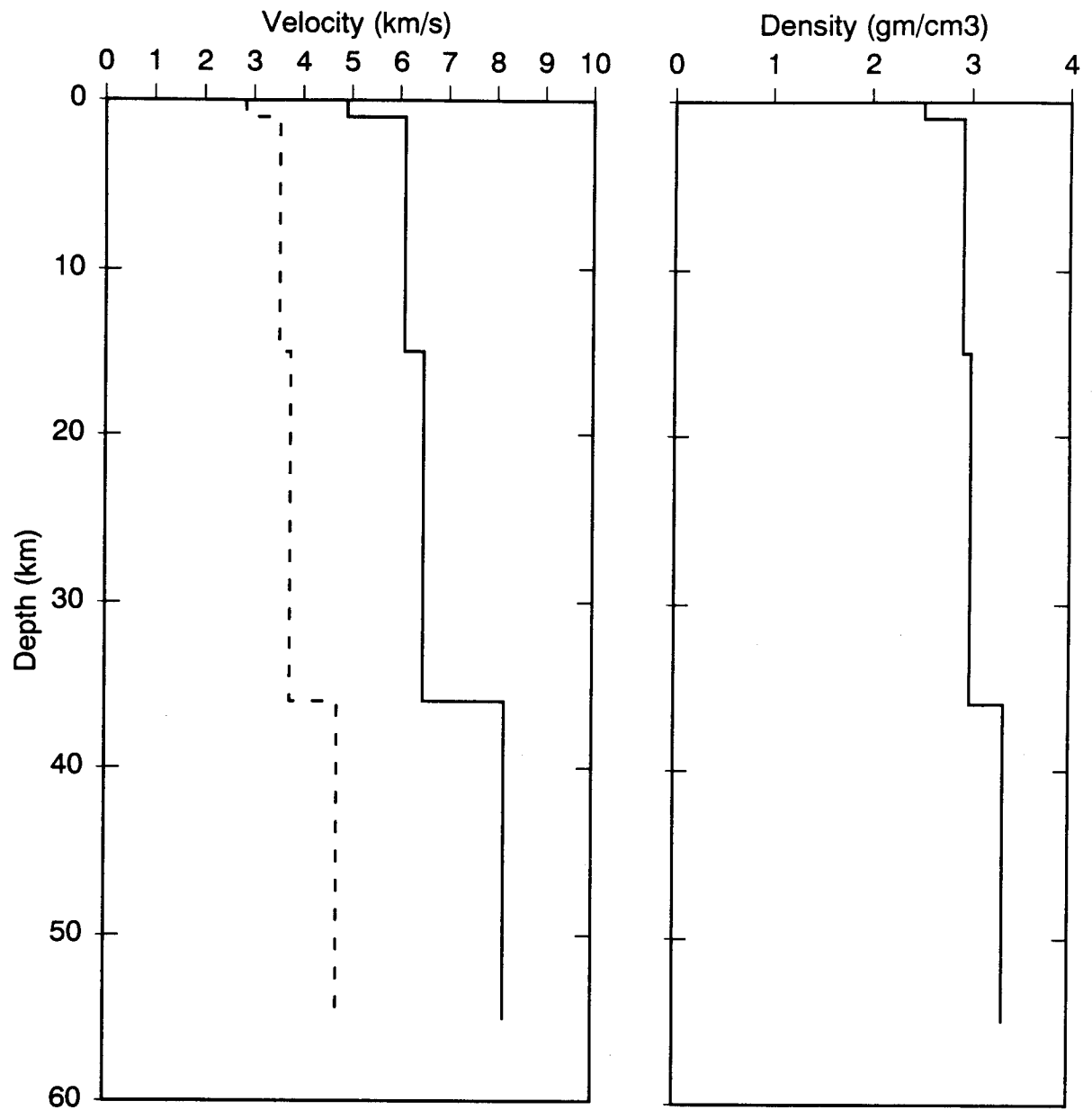


Figure 5.A-3
Crustal model for region 3: Atlantic Coastal Plain.

Region 4: Gulf Coastal Plain

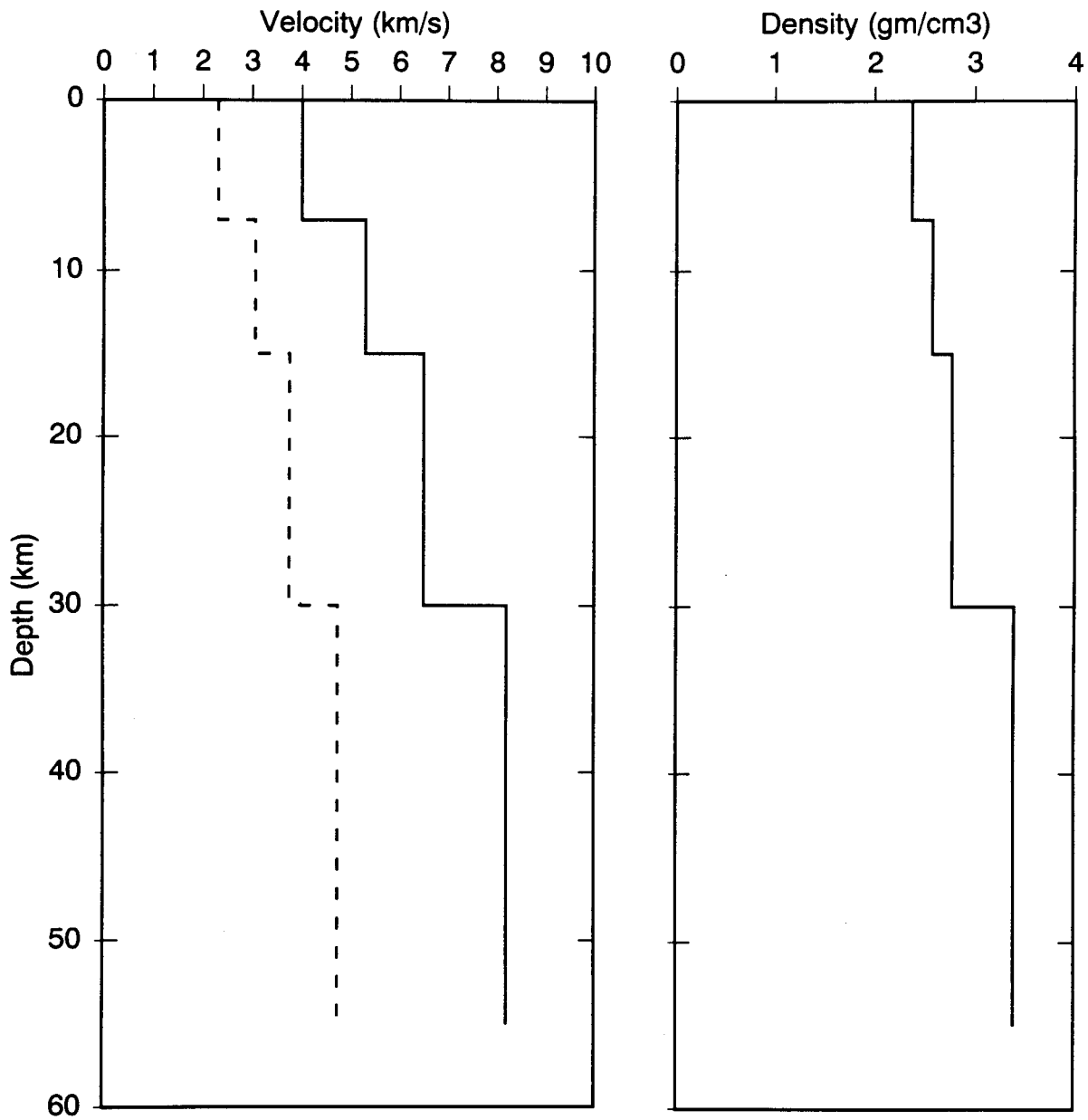


Figure 5.A-4
Crustal model for region 4: Gulf Coastal Plain.

Region 5: Southern Appalachians

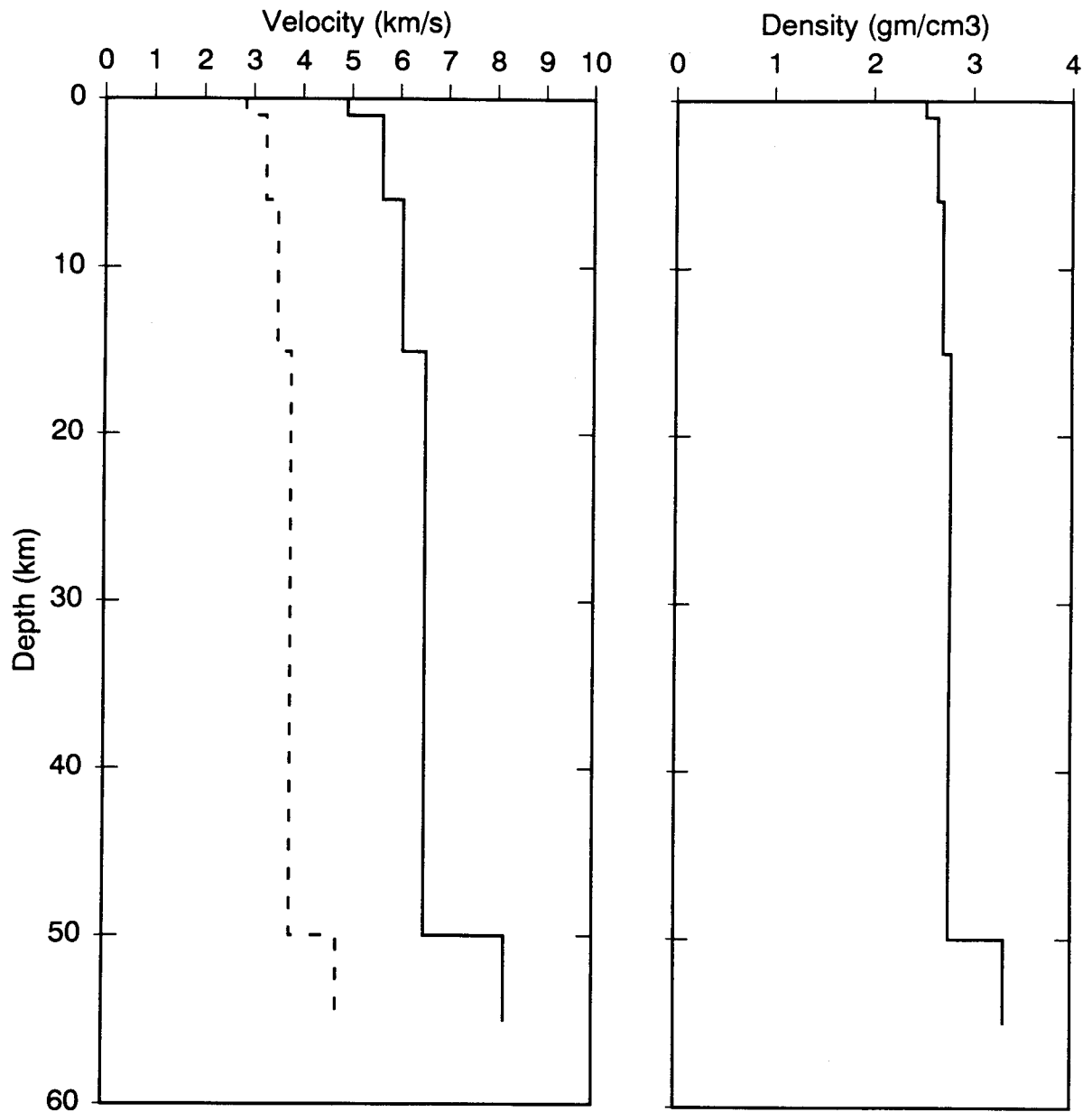


Figure 5.A-5
Crustal model for region 5: Southern Appalachians.

Region 6: Central Tennessee

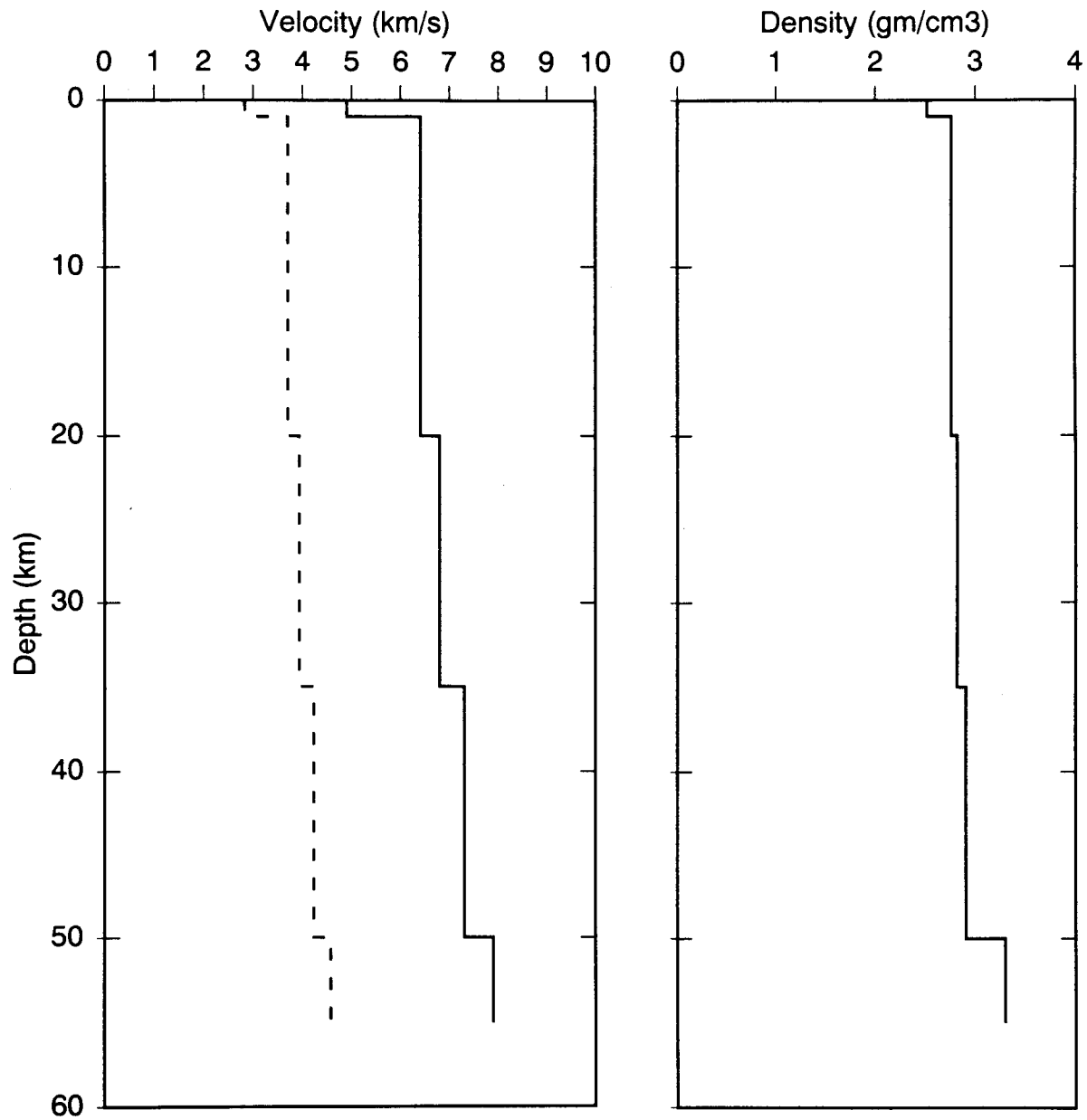


Figure 5.A-6
Crustal model for region 6: Central Tennessee.

Region 7: Western Tennessee

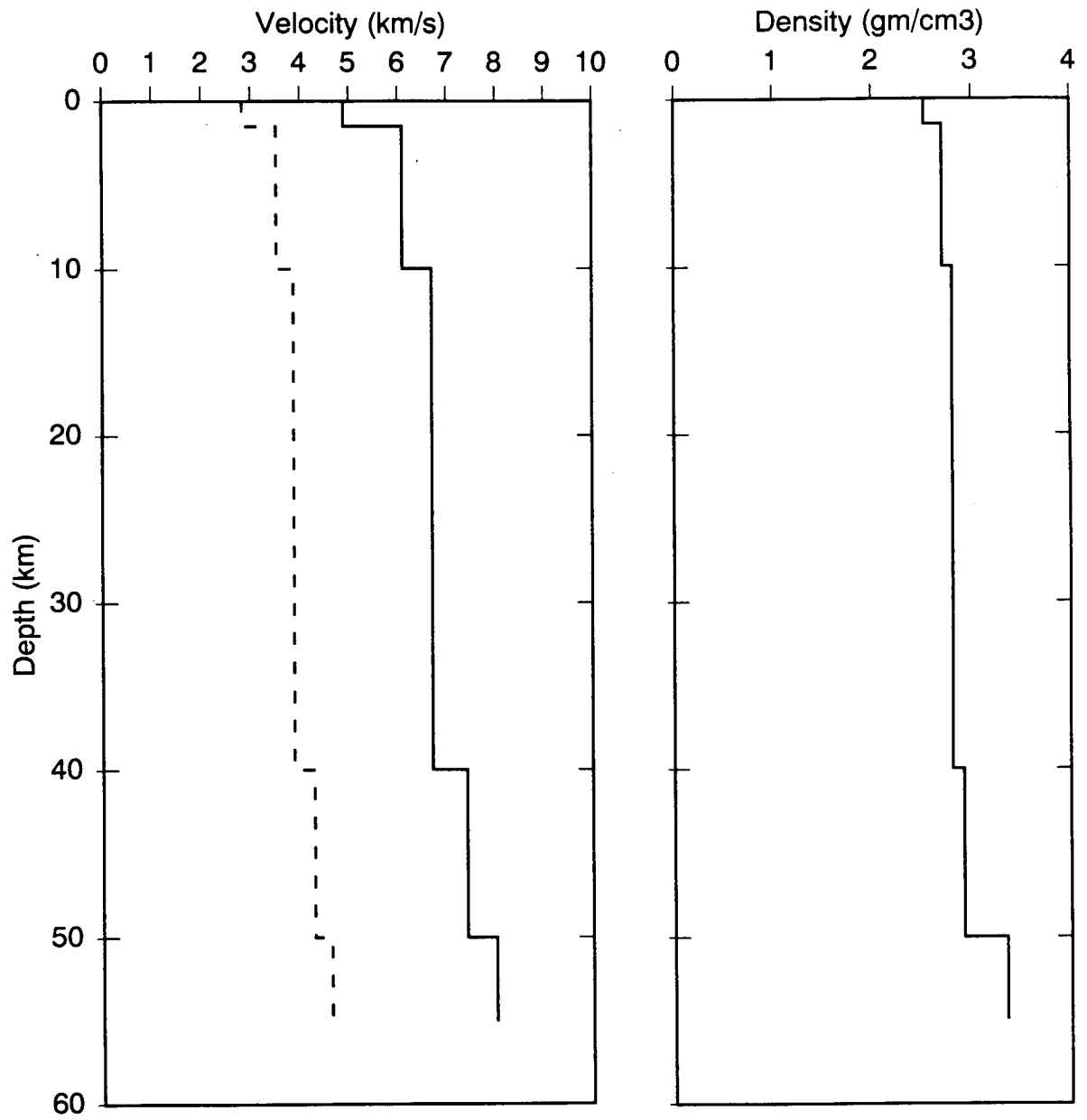


Figure 5.A-7
Crustal model for region 7: Western Tennessee.

Region 8: New Madrid Rift

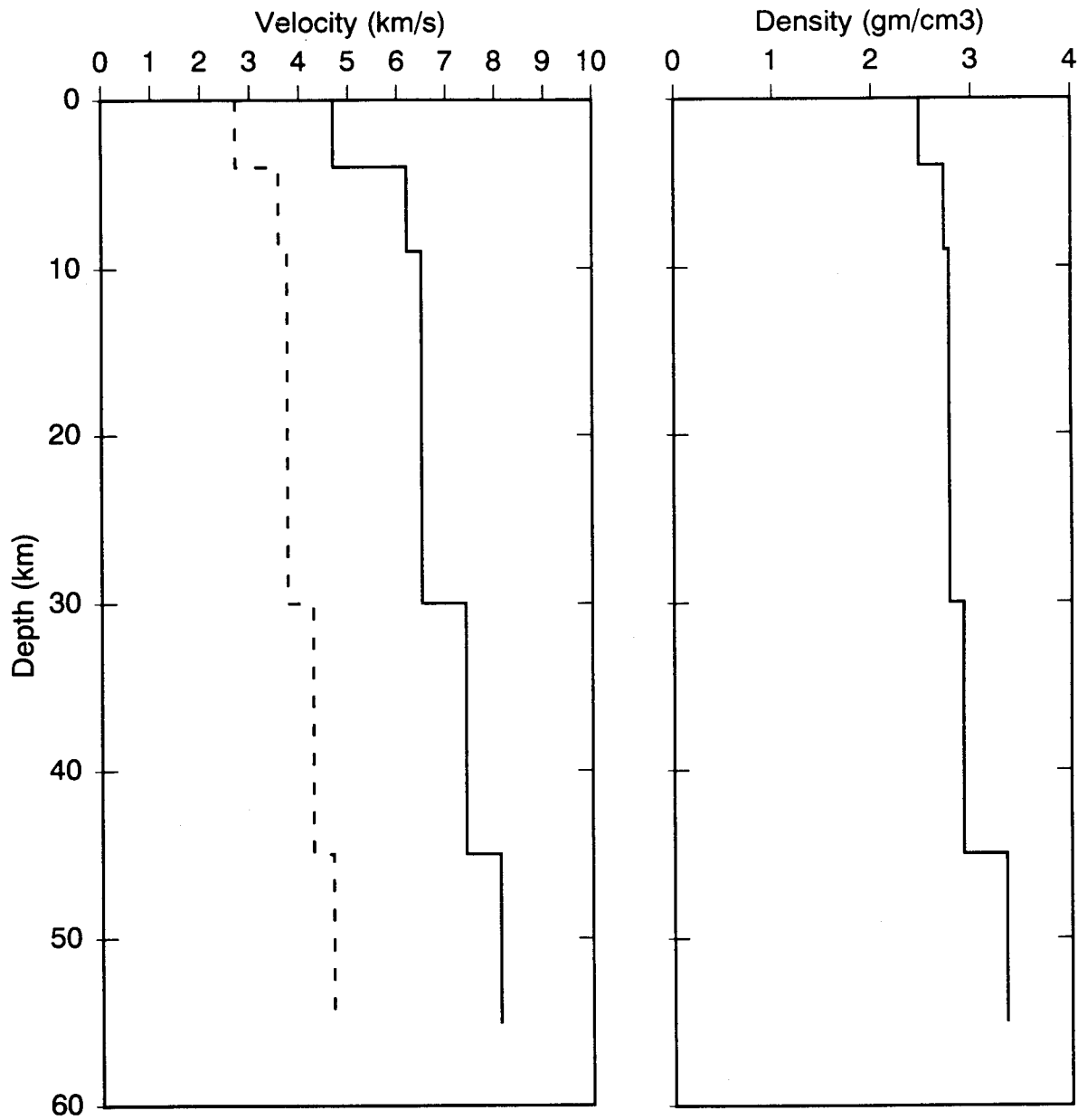


Figure 5.A-8
Crustal model for region 8: New Madrid Rift.

Region 9: Ozarks

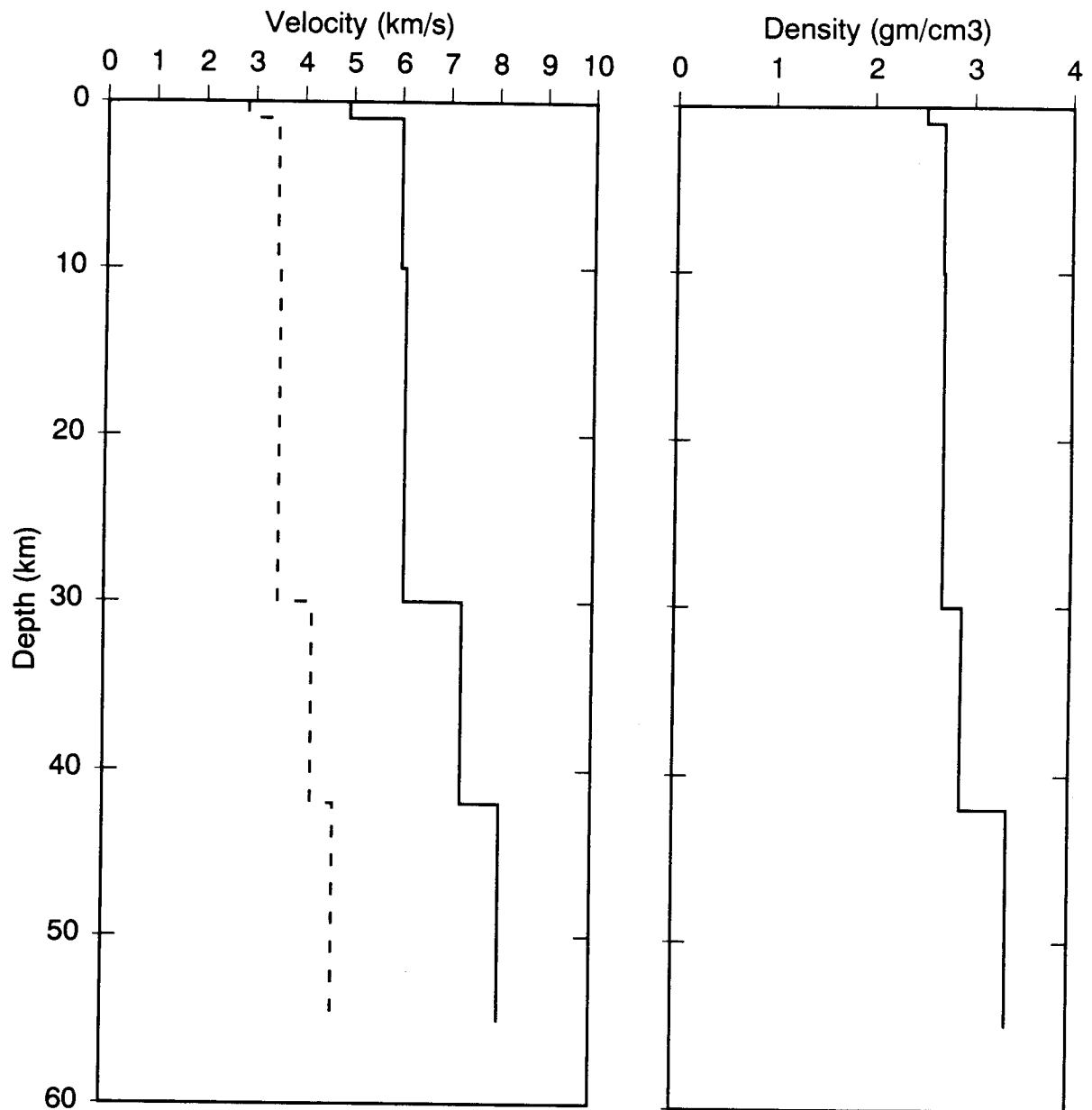


Figure 5.A-9
Crustal model for region 9: Ozarks.

Region 10: Northern Grenville-Superior

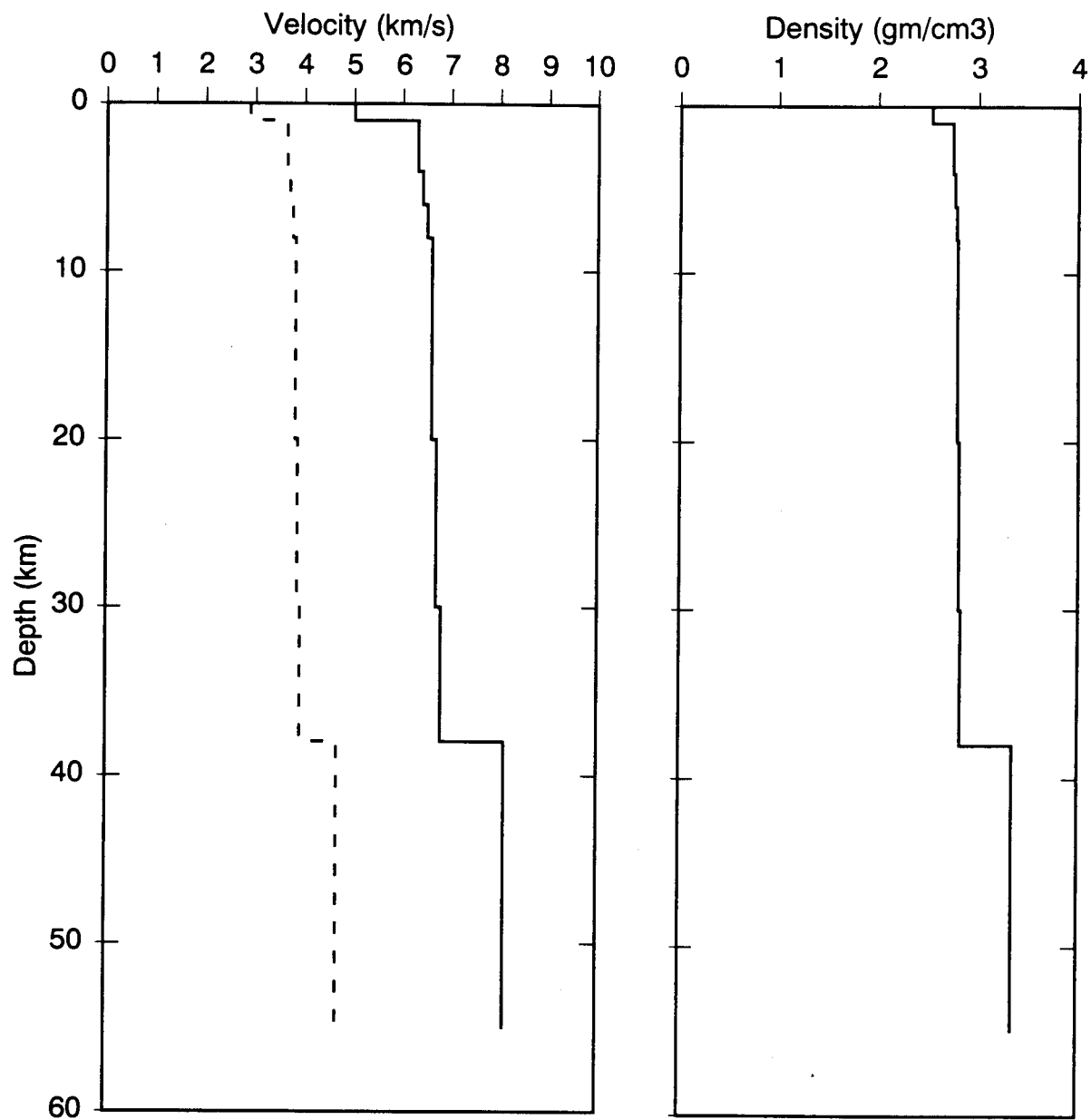


Figure 5.A-10
Crustal model for region 10: Northern Grenville-Superior.

Region 11: Lake Superior Basin

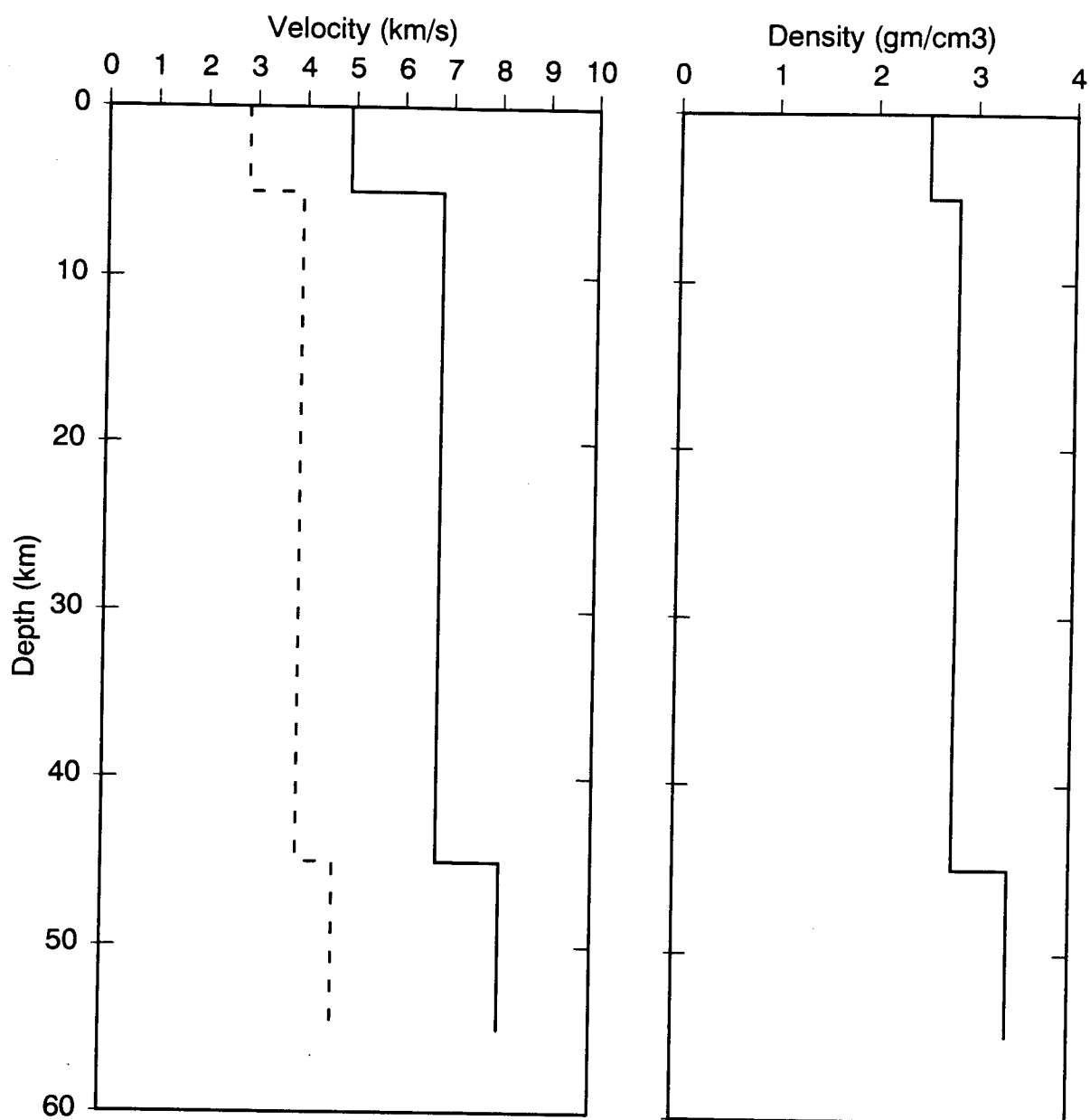


Figure 5.A-11
Crustal model for region 11: Lake Superior Basin.

Region 12: Mid-continent

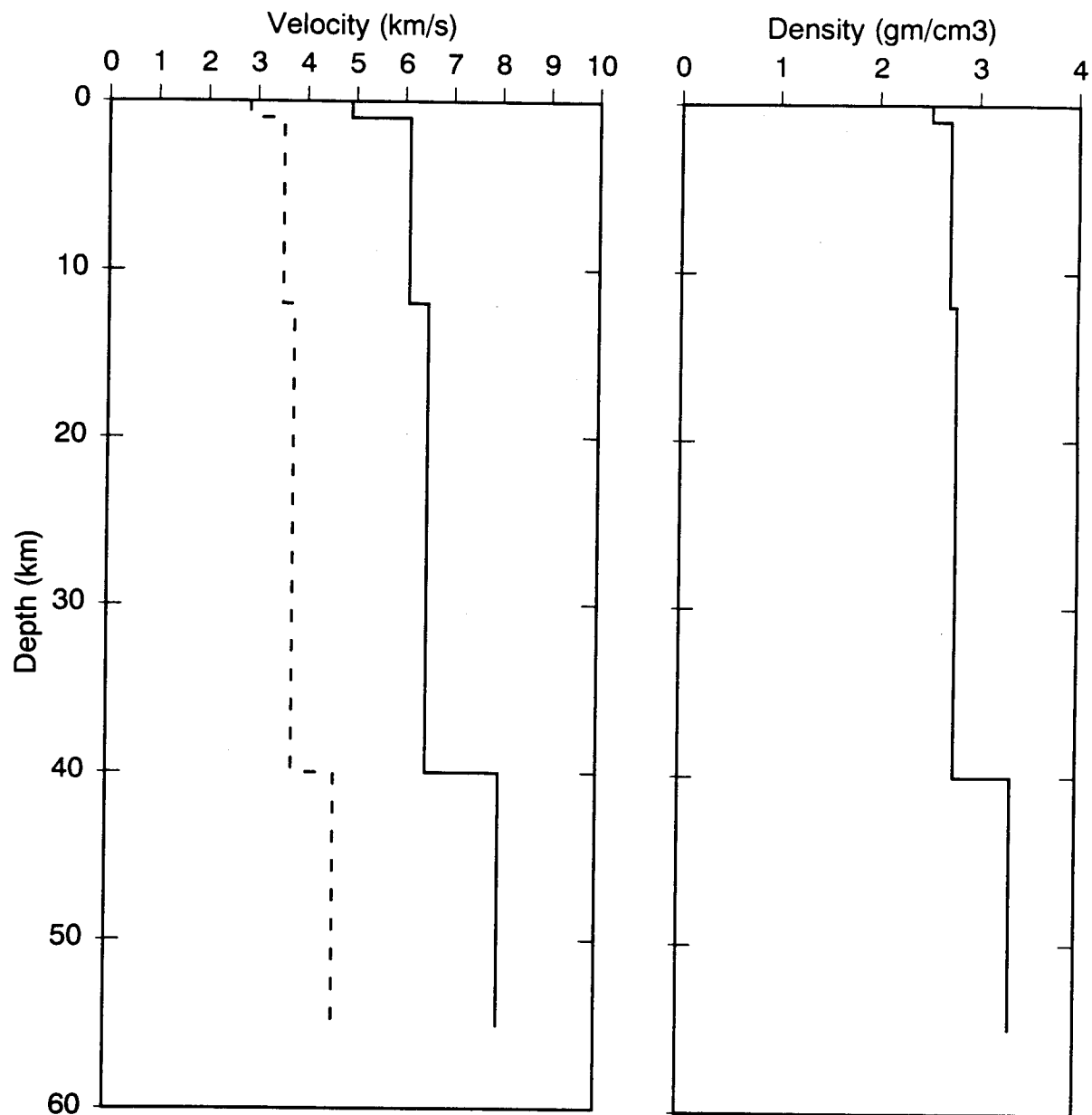


Figure 5.A-12
Crustal model for region 12: Mid-continent.

Region 13: Northern Great Plains

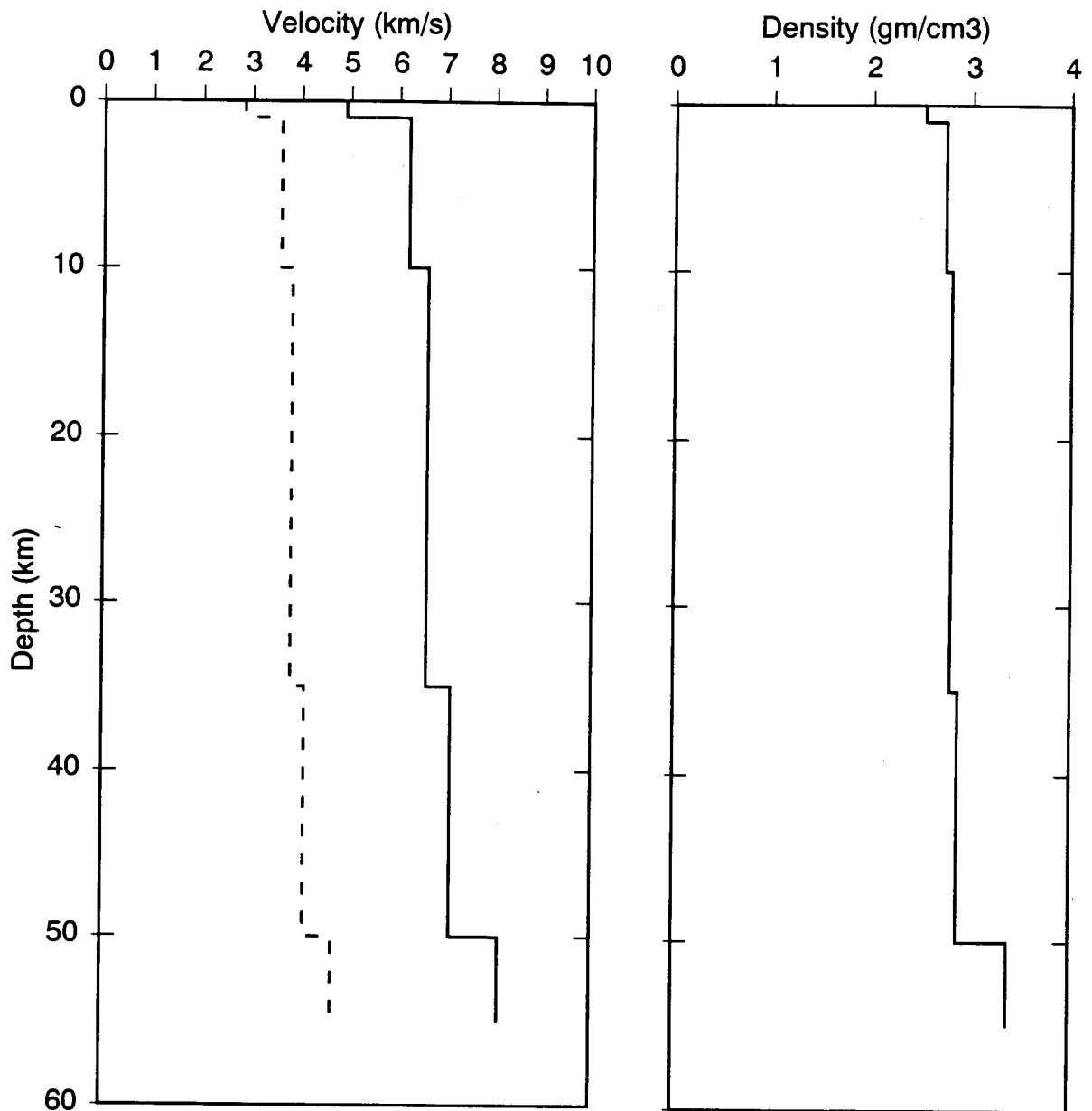


Figure 5.A-13
Crustal model for region 13: Northern Great Plains.

Region 14: Central Plains

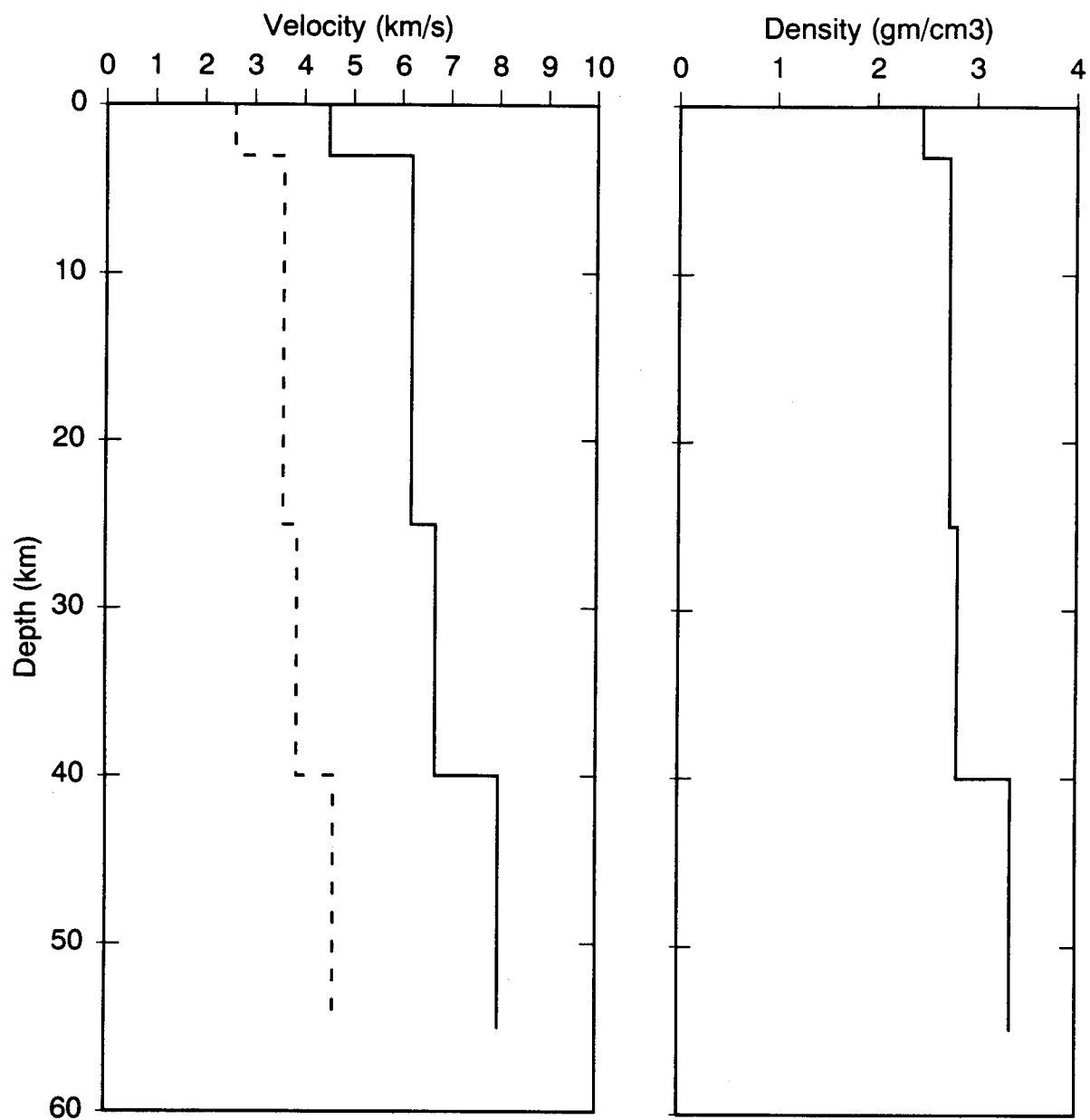


Figure 5.A-14
Crustal model for region 14: Central Plains.

Region 15: Southern Great Plains

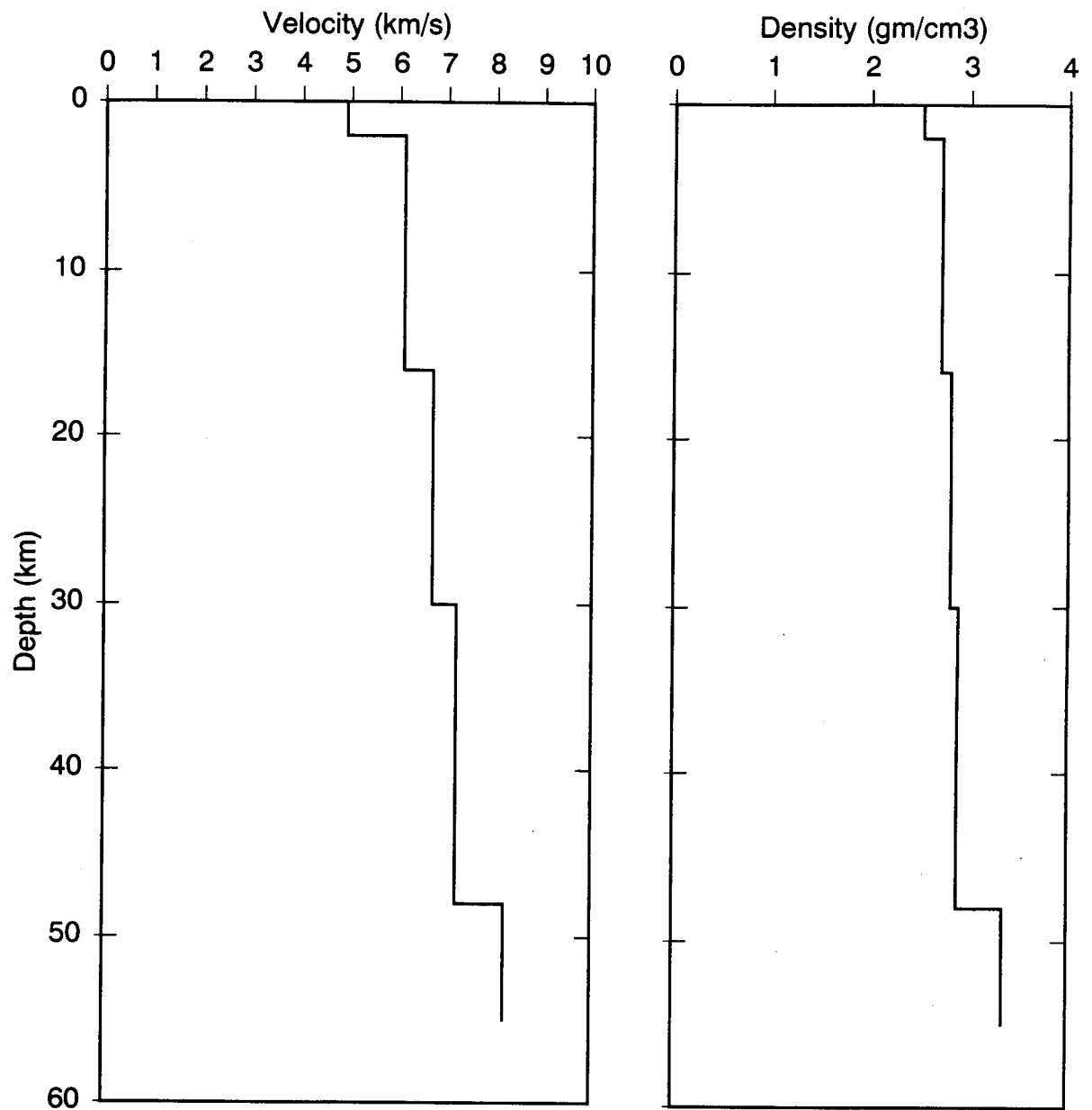


Figure 5.A-15
Crustal model for region 15: Southern Great Plains.

Region 16: Williston Basin

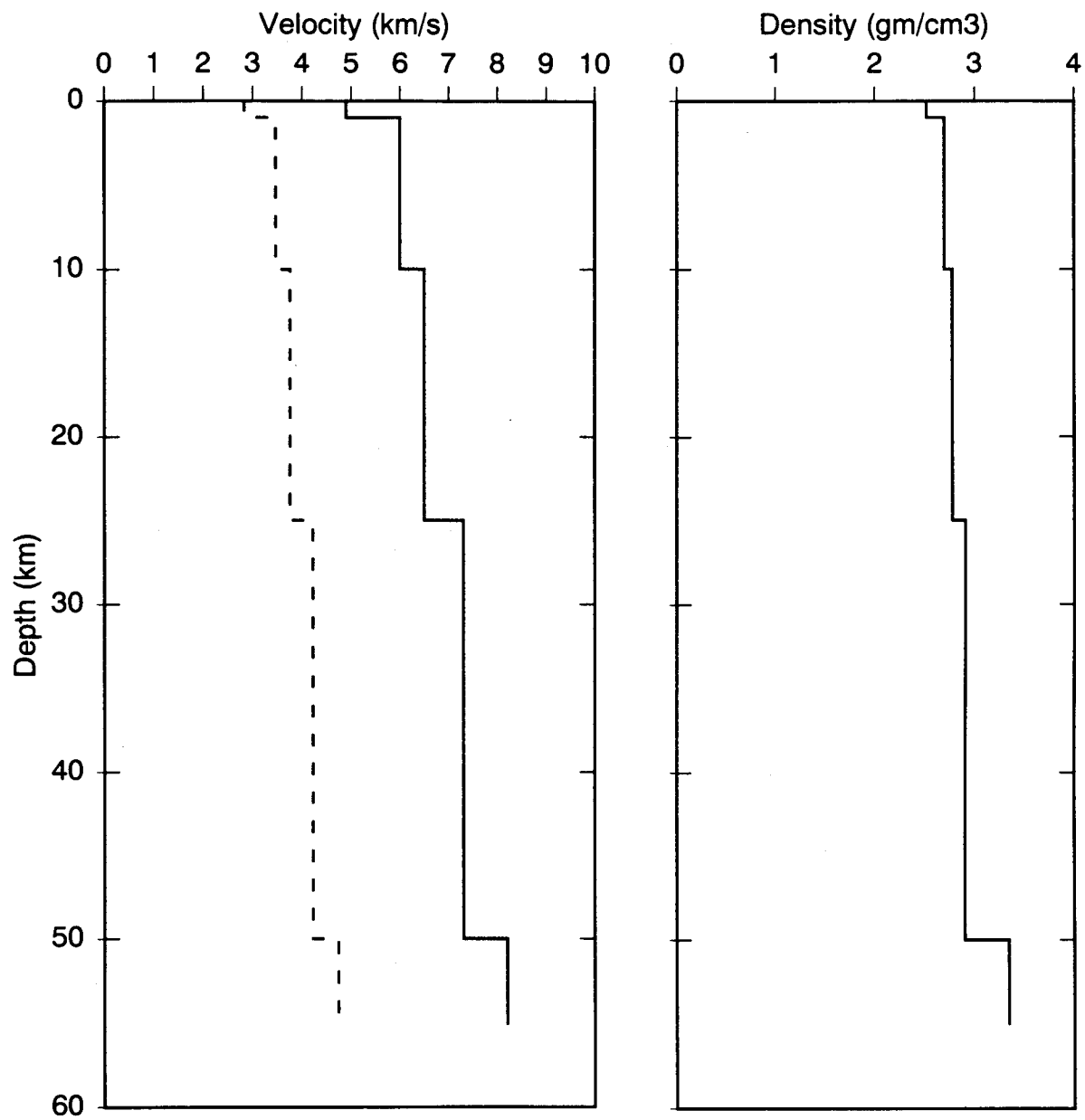


Figure 5.A-16
Crustal model for region 16: Williston Basin.

APPENDIX 5.B

DEPTH DISTRIBUTION MODELS

The development of the focal depth distribution models described in Section 5.1.3 and 5.5 is given in this Appendix.

The rate of seismicity in ENA is low and the catalog of moderate to large magnitude earthquake in ENA with known focal depth is sparse. Therefore, we consider earthquakes in other analogous tectonic regions to provide a larger data set with greater statistical significance. ENA is classified as a Stable Continental Region (SCR). Other SCR regions include parts of Africa, Asia/Russia, Australia, Europe, India, and South America.

The focal depth distributions are developed using the EPRI SCR Earthquake Catalog (Johnston, 1993). All events in the EPRI data base of magnitude 5 or larger having an assigned tectonic type are considered. For events with multiple depth assignments, the preferred depth is used. Of the six SCR regions, only Africa and Australia had earthquake depth distributions similar to that of ENA. The depth distributions of these three continents and their combined depth distribution are shown in Figure 5.B-1. The combined distribution of Eastern North America, Africa, and Australia will be referred to as EAA.

The characteristics of earthquake focal depths in eastern North America have been summarized by Wheeler and Johnston (1993) as follows. The depths of these earthquakes range from 3 to 32 km. There may be systematic variations of focal depth with geological terrain. The deeper events may indicate the presence of mafic or granulite rocks, whereas unusually shallow large earthquakes have only been observed in the central craton. In the central craton, earthquake depths are concentrated at about 5 km, with a range of 3 to 18 km. In the New Madrid region, focal depths are concentrated at about 7 km with a range of 5 to 12 km. Appalachian earthquakes tend to be shallower (in the depth range of 5 to 8 km) than those of comparable size in the late Proterozoic continental margin that underlies the borders of the northwest Appalachian orogen (in the depth range of 8 to 14 km). Deep earthquakes (> 25 km) have only been observed beneath the late Proterozoic margin.

Based on the apparent correlation between focal depth and geologic terrain, the depth distributions for the ENA and EAA are subdivided according to tectonic type: margin, rift, or other. Margins include all crust between the landward limit of normal faulting or significant tectonic thinning and the continental/oceanic transition zone. Rifts are zones of large-scale normal faulting embedded within continental crust and include all crust that lies within the outermost rift-related normal faults. Regions of stable continental crust that are neither margins nor rifts are classified as "other" regions. The depth distributions for EAA are shown in Figure 5.B-2. The depth distributions are tabulated in Table 5.B-1.

5.B.1 Dependence on Tectonic Type

Using the combined EAA data set, the median depths for rifts, margins, and other regions are 10 km, 15 km, and 10 km, respectively. The differences in the medians for margins and "other" events are significant at the 95% confidence level. Due to the small number of rift events (12), the median depth for rifts is not significantly different from the median depth for either margins or "other" regions at the 95% confidence level. The significant difference in the median depths between margins and "other" regions indicates that two depth distributions are justified. Since the median depth for rifts is the same as for "other" regions, the depth distribution for rifted regions is combined with the "other" regions.

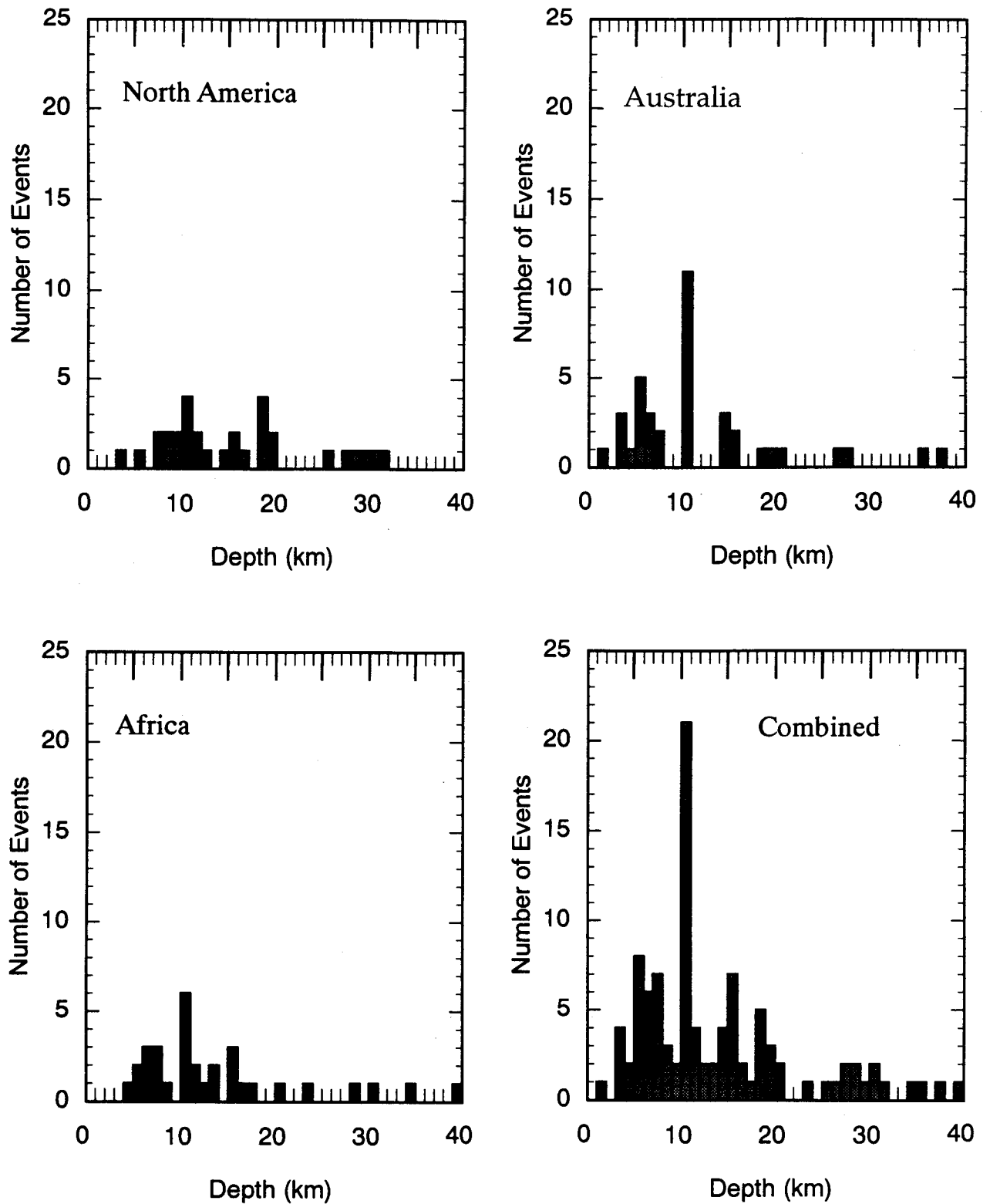


Figure 5.B-1
Focal depth distributions for eastern North America, Australia and Africa from the EPRI Stable Continental Regions Catalog (Johnston et al., 1993).

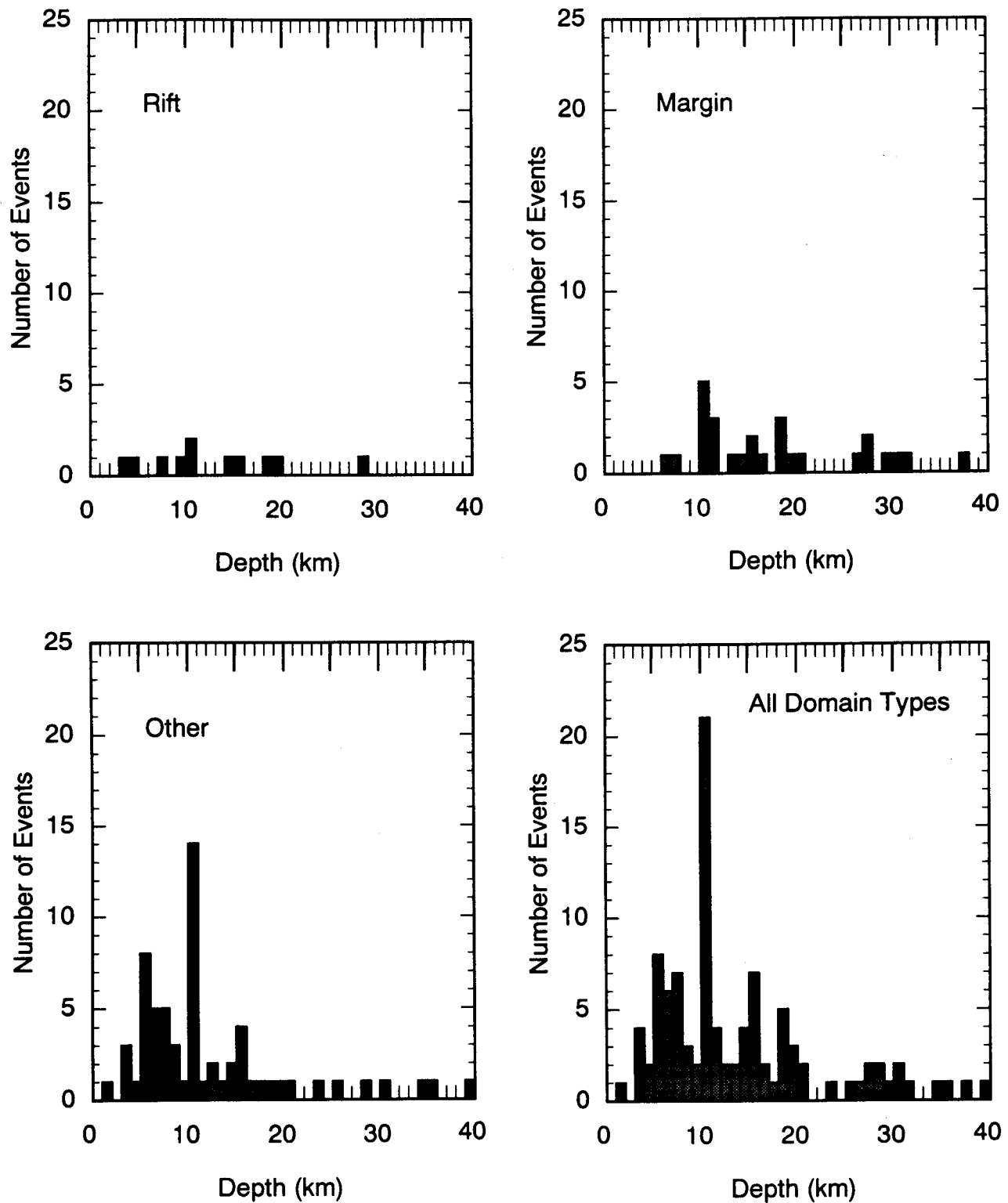


Figure 5.B-2

Separation of focal depths in the combined eastern North America, Africa, Australia data set according to tectonic type: rift, margin, and other. The distribution for margins has a deeper median depth than for rifts or other regions.

Table 5.B-1

Depth Distributions of Earthquakes in Eastern North America (ENA) and Eastern North America, Australia, and Africa (EAA)

Region: Type:	ENA All	EAA All	ENA Rift	ENA Margin	ENA Other	EAA Rift	EAA Margin	EAA Other
DEPTH	No.	No.	No.	No.	No.	No.	No.	No.
1	0	1						1
2	0	0						
3	1	4	1			1		3
4	0	2				1		1
5	1	8			1			8
6	0	6					1	5
7	2	7		1	1	1	1	5
8	2	3			2	0		3
9	2	2	1		1	1		1
10	4	21			4	2	5	14
11	2	4		2			3	1
12	1	2			1			2
13	0	2					1	1
14	1	4			1	1	1	2
15	2	7	1		1	1	2	4
16	1	2		1			1	1
17	0	1						1
18	4	5		3	1	1	3	1
19	2	3	1	1		1	1	1
20	0	2					1	1
21	0	0						
22	0	0						
23	0	1						1
24	0	0						
25	1	2			1		1	1
26	0	2					2	
27		1		0		1		
28	1	3			1	1	1	1
29	1	1		1			1	
30	1	2		1			1	1
31	1	0		1				
32	0	0						
33		0						
34		1						1
35		1						1
36		0						
37		1					1	
38		0						
39		0						
40		0						
41		1						1
42		0						
43		1						1

Using the ENA data set alone, the median depths for margins and non-margins regions are 18 km and 10 km, respectively. The difference in the median depths is consistent with the EAA data set with margin events being deeper, but the difference is not significant at the 95% confidence level due to the small number of events in the ENA catalog.

5.B.2 Comparison of ENA and EAA Focal Depth Distributions

As a check of the consistency between the ENA focal depths and the EAA focal depths, the median and shape of the distributions are compared. The difference in median depths between the ENA and the combined EAA data sets is not significant at the 95% confidence level for either margins or non-margin regions. The shapes of the depth distributions for the ENA and EAA data sets are compared by empirical quantile-quantile plots shown in Figure 5.B-3. If the data points fall along the 45° line, the depth distributions would be statistically identical. For margins, the shapes of the two distributions are similar (close to 45° slope) but with different medians (shifting the points below the line). As noted above, the difference in median depths is not statistically significant. For non-margin regions, the shapes of the distributions are similar for depths up to 25 km (points close to the 45° line), but at larger depths, the tail of the EAA distribution is elongated in comparison to the tail of the ENA distribution (deviation from the line). In comparing the ENA and EAA distributions we found that the larger EAA data set is applicable to the ENA at all focal depths for margins and at focal depth less than 30 km for non-margins. Therefore, the EAA depth distribution for non-margins is truncated at 30 km for application to the ENA. The resulting depth distributions smoothed over 5 km distance bins are shown in Figure 5.B-4. The corresponding normalized depth weights for the two tectonic categories are listed in Table 5-9 along with a generic model which combines both categories.

5.B.3 Alternative Focal Depth Distribution Models

Focal depth distribution models for ENA were presented in Section 5.2.3 for the combined ENA, Australia, and Africa Stable Continental Regions (SCR). For this combined data set, denoted EAA, two focal depth distribution models were given: one model distinguished between margins and non-margins, the other model was generic and did not distinguish by tectonic type (Tables 5-9 and 5-10). These Tables are repeated here as Tables 5.B-2 and 5.B-3. In this appendix, alternative depth distribution models for ENA are presented. These alternative models are used in Section 5.5 to test the robustness of the ground motion regionalization.

Table 5.B-2
Weights for the EAA Depth Distribution Model

Depth	Margins	Non-Margins
5	0.077	0.352
10	0.308	0.338
15	0.192	0.155
20	0.192	0.070
25	0.115	0.042
30	0.115	0.042

Table 5.B-3
Weights for the Generic EAA Depth Distribution Model

Depth	Weight
5	0.077
10	0.308
15	0.192
20	0.192
25	0.115
30	0.115

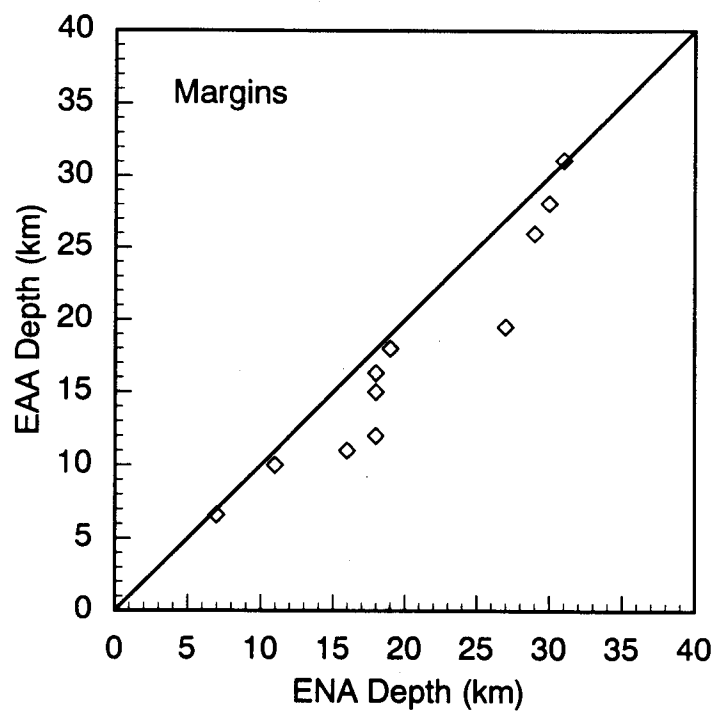
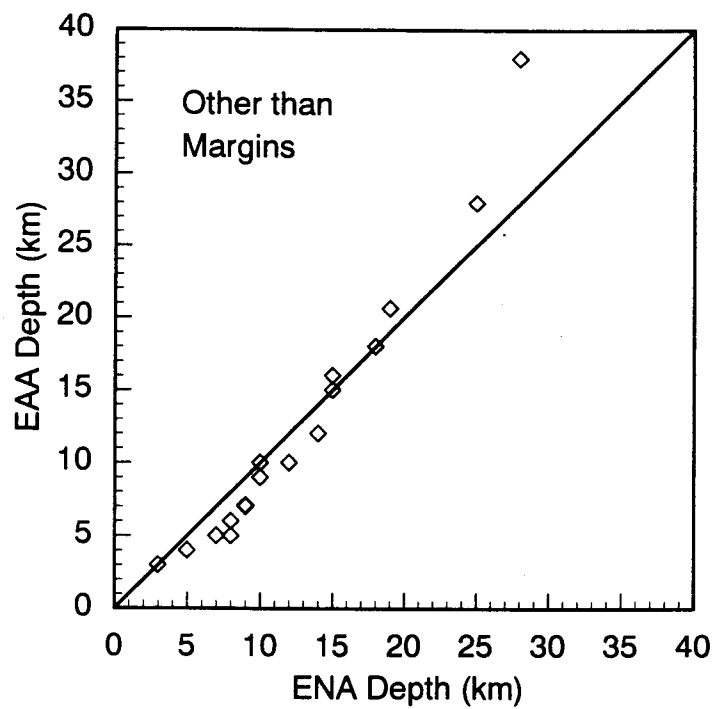


Figure 5.B-3.

Quantile-quantile plot comparing of the depth distribution for eastern North America data set (ENA) with that of the combined eastern North America, Africa, and Australia data set (EAA). A slope of 45° indicates that the distributions have the same shape.

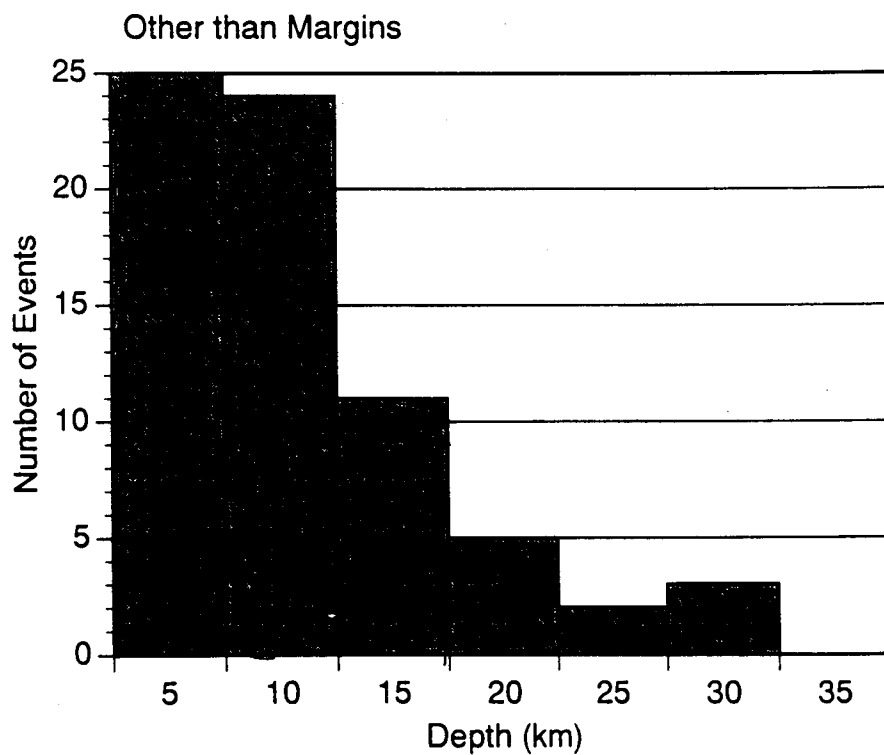
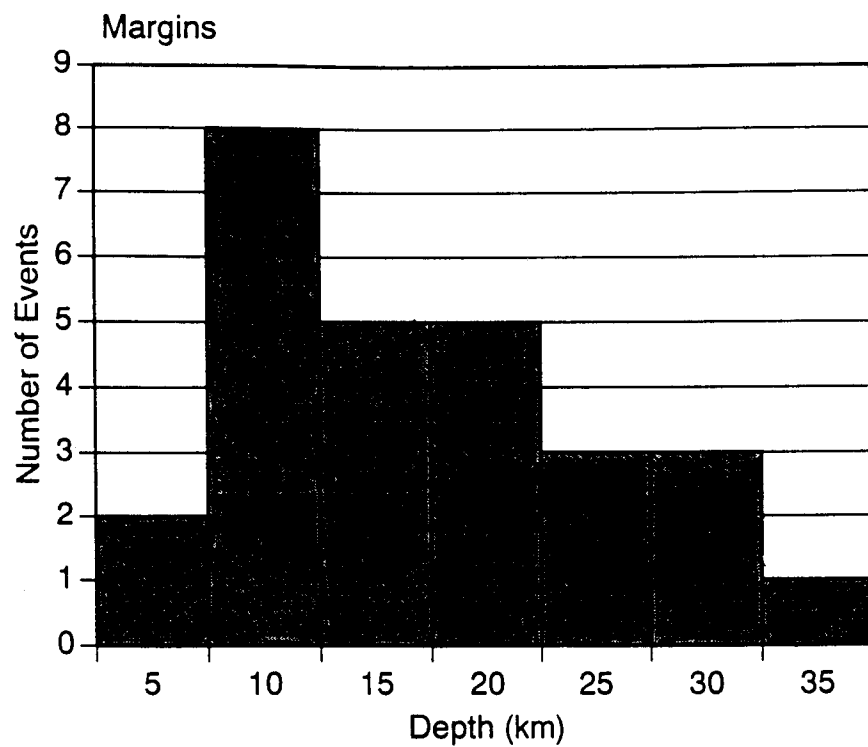


Figure 5.B-4

Depth distribution model for margins and non-margins based on the combined eastern North America, Africa, and Australia data set. This depth distribution model is considered in the regression analysis in Section 9.

As the first alternative model, the depth distribution of ENA alone, excluding the events in Australia and Africa is listed in Table 5.B-4. The ENA data set is too sparse to distinguish between margins and non-margins so only the generic model is given.

Table 5.B-4

Weights for the Alternate Depth Distribution Model (Generic ENA)

Depth	Weight
5	0.174
10	0.478
15	0.174
20	0.261
25	0.043
30	0.174

A second alternative model is based on the Wheeler and Johnston (1993) data set. They selected a subset of 20 of the 31 events of magnitude 5 or larger in ENA from the EPRI ENA catalog. The seven criteria for choosing the subset of events should result in their focal depths being more reliable. The depth distribution of this subset is listed in Table 5.B-5. The main difference between this "high quality" data set and the full ENA data set is the elimination of most of the events at depths of 15–20 km.

Table 5.B-5

Weights for the Alternate Depth Distribution Model (Wheeler and Johnston)

Depth	Weight
5	0.250
10	0.500
15	0.050
20	0.050
25	0.015
30	0.135

The third and fourth alternative depth models are based on a data set of small magnitude events (Atkinson, personal communication). High quality events primarily from the ECTN are used. She found that there is a significant difference between the focal depths for events in rifted versus non-rifted regions with rifted regions having shallower focal depths than non-rifted regions. Therefore, the events were separated into rifted and non-rifted subsets. The depth distributions for rifted regions and non-rifted regions are listed in Table 5.B-6. Many of the 16 velocity regions (Figure 5-12) are partially rifted. The depth distribution model depends on how these partially rifted regions are classified. Two alternative classifications have been used. In one case all of the partially rifted regions are classified as rifted and in the other case all of the partially rifted regions are classified as non-rifted. The partially rifted regions are 2, 10, 12, 14, and 15. Regions 8 and 11 are classified as rifted in both cases. The weights for these depth distribution model are given in Table 5.B-6.

Table 5.B-6

Weights for the Alternative Depth Distribution Model (Atkinson)

Depth	Rifted	Non-Rifted
5	0.17	0.64
10	0.38	0.20
15	0.22	0.10
20	0.16	0.06
25	0.06	0.00
30	0.02	0.00

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APPENDIX 6.A

PROBABILISTIC MODEL OF SOIL-PROFILE VARIABILITY

6.A.1 Introduction

This appendix presents the probabilistic characterization of the possible shear-wave velocity profiles at a candidate soil site for which no site-specific velocity information is available (i.e., a generic soil site). The resulting probabilistic model is then used to generate a suite of artificial profiles, for which site-amplification analyses are then performed (Section 6.A.2) and statistics are computed on the calculated site-amplification factors.

Two approaches have been used in the past to calculate the effect of soil-profile variability on site-amplification factors. The first approach varies the velocities at all depths assuming perfect correlation, by either multiplying the velocities at all depths by a random factor (e.g., McGuire et al., 1988), or by adding a random quantity to the velocities at all depths (Toro et al., 1992). The second approach assumes independence between the velocities in adjacent layers (Costantino et al., 1991). Both approaches represent extreme, albeit convenient, assumptions about the roughness of real soil profiles. The model developed here represents a mid-point between the two extreme models described above. The model functional form and parameters have been obtained using data heretofore unavailable. This model will lead to more realistic quantification of soil amplification and its variability in generic soil profiles.

Data for the development of this model come from an extensive database of soil profiles compiled under this study. This appendix documents the selection of data for analysis (Section 6.A.2), the development and validation of the probabilistic model, and the generation of sample artificial profiles (Section 6.A.3).

In addition, this appendix presents a procedure to modify the generic model in order to generate site-specific artificial profiles (Section 6.A.4). This procedure incorporates both site-specific profile information and the parameters of the generic model.

6.A.2 Data

The EPRI soil-profile database currently contains information on 411 soil profiles. Some of the most important profiles in the database come from recent site-characterization studies, as follows:

- The Lotung, Taiwan, experiment.
- The Savannah River site, Aiken, South Carolina.
- The EPRI-sponsored work at Gilroy 2 and Treasure Island.
- The USGS effort to characterize site response at stations that recorded the 1987 Loma Prieta earthquake.

Additional information on this studies is provided in Section 8. A detailed description of the database is provided in Silva et al. (1993, in preparation).

For the purposes of this analysis, we utilized only those profiles that satisfied the following conditions:

- The profile contains shear-wave velocity information based on field measurements.
- The profile contains interpreted velocities (as opposed to raw velocities).
- The velocity measurements were made during or after 1974. Many site investigations made after 1974 followed the guidelines for site investigation contained in the "Appendix A" reactor-siting criteria (Atomic Energy Commission, 1973), making them more reliable.
- The site is classified as soil. Rock and till sites are excluded. As a result, the model applies only to "sand-like" sites.

A total of 218 profiles meet these requirements. Most of these profiles come from California sites. A list of these profiles is contained in Table 6.A-1.

Some profiles contain a description of material type as a function of depth, others do not. If the lower portion of a profile consists of rock, as determined from the soil description, this portion is excluded from the analysis. In Table 6.A-1, the "Profile Depth" entry indicates the total depth of the profile. The "Rock Depth" entry indicates the depth to rock, if available, determined as indicated above. The "Depth" entry indicates the profile depth considered in the analysis.

6.A.3 Data Analysis, Fitting of Model, and Results

Figure 6.A-1 shows all profiles selected for this analysis. This figure gives an indication of the variability in velocities. Figures 6.A-2 and 6.A-3 show samples of profiles from categories III and IV (corresponding to depths of 80-180 ft and 180-400 ft, respectively; see Section 6.2.2 for definition of depth categories). Figures 6.A-4 and 6.A-5 show the logarithmic mean $\pm$ standard-deviation ranges of profiles in these same categories; Figure 6.A-6 shows the same information for all categories combined. Figures 6.A-4 through 6.A-6 (especially Figure 6.A-4) show that the standard deviation tends to increase with mean velocity, suggesting a model where the coefficient of variation (COV, i.e., the ratio of standard deviation to mean) remains constant. They also show correlation among layer velocities, in the sense that a profile that starts with above-average velocity has a tendency to remain above average.

The probabilistic model to characterize these profiles attempts to capture the main features observed in Figures 6.A-1 through 6.A-5; namely, random layering, velocities with a constant COV, and correlated velocities. Figure 6.A-7 illustrates the two elements of the soil-profile model. The layering model describes the thicknesses of the layers (or the location of layer boundaries). The velocity model describes the velocity at the mid-point of each layer.

Regarding the layering, we note that layers tend to be thinner near the surface and thicker at depth (see Figures 6.A-1 through 6.A-4). A simple probabilistic model to characterize the layering is a Poisson process with depth-dependent rate (i.e., a non-homogeneous Poisson process; see Parzen, 1962). In this model, the mean layer thickness is depth-dependent, but the thickness of layer i is probabilistically independent of the thickness of layer $i-1$. In a Poisson model with constant rate, the layer thicknesses follow an exponential distribution. Because we utilize a depth-dependent rate, the distribution of layer thickness is no longer exponential, but this causes no problems in the analysis or in the generation of artificial profiles.

We adopt a modified power-law model to characterize the depth-dependent rate of layer boundaries, i.e.,

$$\lambda(h) = c_3 [h + c_1]^{-c_2} \quad (6.A-1)$$

where λ is the rate of layer boundaries (foot⁻¹) and h denotes depth in feet. Coefficients c_1 through c_3 are estimated from the data, using the method of maximum likelihood (Benjamin and Cornell, 1971). The following result is obtained

$$\lambda(h) = 4.86[h + 112]^{-1.03}. \quad (6.A-2)$$

Figure 6.A-8 compares the rate function in Equation 6.A-2 to the observed rate of layer boundaries¹ (or transition rate). The observations are shown as running averages over a 33 ft window (i.e., the quantity shown for a depth of 150 ft is the number of layer boundaries between 134 and 166, divided by the number of profiles with depths greater than or equal to 50 m). This figure shows a good agreement between the data and the fitted model. In addition, a statistical test for Poisson (Parzen, 1962) indicates no significant deviation from the assumption of a Poisson process.

¹It is useful to think of the rate of layer boundaries as the reciprocal of the mean layer thickness. This is strictly applicable only if the rate varies slowly.

The above function was modified so as to predict a mean layer thickness of approximately 12 feet near the surface, based on the observation that surficial layers are typically thinner than predicted by Equation 6.A-2. This observation is not reflected in the data because the first few thin layers are often merged into one layer in interpreted profiles. The resulting equation is of the form:

$$l(h) = \max \left\{ \begin{array}{l} 4.68[h + 112]^{-1.03} \\ 0.082 \exp \left[-\frac{1}{2} \left(\frac{h}{26} \right)^2 \right] \end{array} \right\} \quad (6.A-3)$$

which is shown as a heavy solid line in Figure 6.A-8.

As shown in Figure 6.A.7, the velocity model operates on the velocities at the layer midpoints. One can think of these velocities, for a given profile, as a sequence of velocity values (i.e., $V_1, V_2, V_3, \dots$). The velocity model defines the probability distribution of V_i (or a transformation thereof), and its correlation with the velocities in adjacent layers. Transforming the velocities to $\ln(\text{Velocities})$ has the effect of making the standard deviation of the transformed set nearly independent of depth (because the velocities have a nearly constant COV).

In order to investigate the probability distribution of $\ln(\text{Velocity})$, we show the cumulative distribution of the quantity

$$Z_i = \frac{\ln(V_i) - \ln[V_{\text{median}}(h_i)]}{\sigma_{\ln V}} \quad (6.A-4)$$

using a normal-probability scale for the vertical axis (Figure 6.A-9). The thin solid lines represent the 10% Kolmogorov-Smirnov bounds (i.e., a sample from a log-normal distribution has a 10% probability of having points outside these bounds; see Benjamin and Cornell, 1971). The observed log-velocities plot as a nearly straight line, well within the bounds, indicating that the velocities follow a nearly log-normal distribution.

We characterize the lognormal distribution of velocities and the correlation among layers by means of a first-order auto-regressive model; i.e.,

$$\begin{aligned} Z_1 &= \varepsilon_1 \\ Z_i &= \rho Z_{i-1} + \sqrt{1 - \rho^2} \varepsilon_i, \quad i > 1 \end{aligned} \quad (6.A-5)$$

where ρ is the serial auto-correlation coefficient of Z , and $\varepsilon_1, \varepsilon_2, \varepsilon_3, \dots$ are independent normal random variables with zero mean and unit standard deviation 0.74. We estimate $\sigma_{\ln(\text{Vel.})}$ and ρ using linear regression, obtaining $\sigma_{\ln(\text{Vel.})} = 0.39$ (corresponding to a velocity COV of 41%), and $\rho = 0.577$.

Equation 6.A-5, parameters $\sigma_{\ln(\text{Vel.})}$ and ρ , and a model for the median velocity profile, completely define the probabilistic velocity model. The velocity model, together with the layer model, can be used to generate artificial generic profiles of any desired depth in order to calculate amplification factors. The median velocity profile to be used is based on the profiles at several nuclear-plant sites (see Section 6.2.2).

Examination of artificial profiles models indicates the presence of a few "outlier" profiles, with very high shear-wave velocities (see Figure 6.A-10)². This problem is caused by the heavy upper tails of the lognormal distribution, and can be alleviated by truncating the distribution of residual ε in Equation 6.A-5, as well the distribution of log-velocity, at 2 standard deviations. Figure 6.A-11 shows the

²In Figures 6.A-10 and 6.A-11, the location of layer boundaries calculated by the Poisson simulation algorithm are rounded off to coincide with the depth discretization used in the site-response calculations. This is why the layer boundaries appear more regular on the bottom half of the figures (where the discretization is more coarse).

profiles obtained using truncation. Because this model is intended for the calculation of central tendency and variability in ground-response predictions (rather than for the calculation of extreme values), the treatment of tails has little effect on the results.

Figure 6.A-12 shows five typical profiles generated with the model presented here. These profiles are similar in appearance to the real profiles in Figures 6.A-1 through 6.A-3, especially the latter. Figure 6.A-13 shows the logarithmic mean standard deviation range from 50 artificial profiles. The observed statistics are consistent with those in the model.

The model presented here provides a simple and convenient approach to generate a suite of artificial soil profiles for the calculation of site-response variability in generic soil sites.

6.A.4 Model for Site-Specific Profiles

A site-specific velocity model could be developed by collecting site-specific profile data and repeating the analysis procedure followed in Section 6.A.3. The main features of the resulting model would be a lower velocity COV and a median profile different from the generic median profile developed in Section 6.2.2. This approach is generally not practical because it requires a large number of site-specific profiles. The procedure developed here uses the site-specific velocity measurements to modify the median profile and COV of the generic model. The relative weights given to the site-specific data and to the generic parameters depend on the amount of data and its (profile-to-profile) variability. The emphasis here is on the site-specific median profile and COV, which are the most important elements of the velocity model, rather than on the layering model and the between-layer correlation.

It would be difficult to develop a procedure of this kind that would be applicable to all kinds of sites. The procedure developed here is expected to be applicable for a majority of sites. There will be sites where the procedure needs common-sense modifications (a few obvious examples are provided). Judgment, rather than quantitative tests, is the best tool for determining the applicability of the model or the type of modifications required.

Section 6.A.4.1 documents the assumptions used to modify the parameters of the generic model and the derivation of equations to calculate the site-specific median and variability. Section 6.A.4.2 presents step-by-step instructions for these calculations. Finally, Section 6.A.4.3 discusses some situations where the procedure needs to be modified.

6.A.4.1 Model Assumptions and Derivation

The basic assumption in this formulation is that the log-velocities from all profiles at the site, at a given depth, represent equip-correlated realizations from a normal distribution with parameters given by the generic model presented in Section 6.A.3. An additional assumption is that the correlation coefficient between log-velocities at the site is the same for all depths.

The assumption of equi-correlated log-velocities implies that there are some characteristics in common among the profiles at the site. Therefore, there is less variability among the profiles at the site than among profiles at different locations. A more sophisticated model would have the correlation between two profiles at the site depend on the distance between them.

The assumption of a correlation coefficient that does not vary as a function of depth is dictated by the limited data available. This assumption allows us to use data from multiple depths to estimate one correlation coefficient, thereby obtaining a more stable estimate.

Assume that n velocity profiles have been obtained at different locations in the site³. Assume that one is interested on the velocity at another location within the site (location $n+1$), for which no measurements are available. Not having measurements at location $n+1$, one wishes to calculate the median and COV of velocity at site $n+1$, given the velocities observed in the other n profiles. Considering a certain depth h_j , the first assumption above implies that the vector of normalized log-velocities (see Equation 6.A-4)

³ n is expected to be greater than or equal to 6.

associated with the n observations and the un-observed velocity at location $n+1$ (i.e., the vector $[Z_{1j}, Z_{2j}, \dots, Z_{nj}, Z_{n+1,j}]^T$) follows a joint normal distribution with mean equal to the zero vector and covariance matrix (of dimension $n+1$) equal to

$$\Sigma_z = \begin{bmatrix} 1 & \rho_s & \cdots & \rho_s \\ \rho_s & 1 & & \vdots \\ \vdots & & \ddots & \rho_s \\ \rho_s & \cdots & \rho_s & 1 \end{bmatrix} \quad (6.A-6)$$

where ρ_s ($0 \leq \rho_s \leq 1$) is the correlation coefficient of log-velocities at the site. A high value of ρ_s indicates that the variability among the measured site-specific profiles is much lower than the overall variability in the generic model. The procedure to estimate ρ_s from the measured profiles will be described later. Note that the mean vector is zero and the diagonal elements of the covariance matrix are unity because the log-velocities are normalized with respect to the generic model by Equation 6.A-4 (i.e., the logarithm of the generic median velocity is subtracted and the result is divided by the generic $\sigma_{\ln(\text{Vel.})}$).

Using the properties of the multinormal distribution, one can calculate the conditional mean and variance of the normalized log-velocity at location $n+1$ (i.e., $Z_{n+1,j}$) given the observed log-velocities (i.e., $[Z_{1j}, Z_{2j}, \dots, Z_{nj}]^T$). After some matrix algebra, one obtains the following expressions:

$$\text{mean}(Z_{n+1,j} | Z_{1j}, Z_{2j}, \dots, Z_{nj}) = \frac{1}{1 + \frac{1 - \rho_s}{\rho_s n}} \bar{Z}_j \quad (6.A-7)$$

$$\text{Var}(Z_{n+1,j} | Z_{1j}, Z_{2j}, \dots, Z_{nj}) = 1 - \frac{\rho_s^2 n}{1 + \rho_s(n-1)} \quad (6.A-8)$$

where $\bar{Z}_j$ is the site-specific average log-velocity (from profiles 1 through n) at depth h_j . Note that the weight given to the site-specific average increases, and the conditional variance decreases, as n increases (a larger sample) or as ρ_s increases (recall that a high value of ρ_s is an indication of low variability among the measured site-specific profiles). If n is small or ρ_s is near 0 (i.e., the site-specific measurements have little information content) the conditional mean and variance are not too different from their generic values⁴. These results can be used to calculate site-specific values of the median profile and of $\sigma_{\ln(\text{Vel.})}$. Implementation details are provided below.

6.A.4.2 Implementation

Let n be the number of available site-specific shear-wave velocity profiles. The profiles are discretized into m depths such that V_{ij} represents the velocity in profile i at depth h_j . The discretization interval may be constant (e.g., 3 to 5 ft) or it may increase as a function of depth so that shallower depths are given more weight.

The first step is the calculation of the average and standard deviation of log-velocity among the site-specific profiles. The average is calculated as a function of depth, while the standard deviation is obtained by pooling data from all depths in order to obtain a more stable estimate (at the expense of possible model mis-specification); i.e.,

$$\overline{\ln V_j} = \frac{1}{n} \sum_{i=1}^n \ln V_{ij} \quad j = 1, \dots, m \quad (6.A-9)$$

$$\sigma_s^2 = \frac{1}{m} \sum_{j=1}^m \frac{1}{n-1} \sum_{i=1}^n [\ln V_{ij} - \overline{\ln V_j}]^2 \quad (6.A-10)$$

⁴Note that the generic mean and variance of Z are 0 and 1 because we are working with normalized quantities.

It is useful, as a diagnostic, to calculate and plot the standard deviation as a function of depth. Strong dependence on depth may require modifications to the model (see Section 6.A.4.3).

The second step is the calculation of the site-correlation coefficient ρ_s , which is calculated as

$$\rho_s = 1 - \frac{\sigma_s^2}{0.39^2} \quad (6.A-11)$$

where 0.39 is the value of $\sigma_{\ln(\text{Vel.})}$ in the generic model. An estimate of $\sigma_s > 0.39$ is unlikely in practice, but is possible due to statistical variability or other causes. This situation is further discussed in Section 6.A.4.3.

The third step is the calculation of the conditional mean log-velocity profile and the standard deviation of log-velocity (given the observed profiles), which are obtained from Equations 6.A-7 and 6.A-8; as follows:

$$(\ln V_{\text{median},j})_{\text{site-spec.}} = \frac{1 - \rho_s}{1 + \frac{\rho_s n}{\rho_s n}} \ln[V_{\text{median}}(h_j)] + \frac{1}{1 + \frac{1 - \rho_s}{\rho_s n}} \overline{\ln V_j} \quad j = 1, \dots, m \quad (6.A-12)$$

$$\sigma(\ln V)_{\text{site-spec.}} = 0.39 \sqrt{1 - \frac{\rho_s^2 n}{1 + \rho_s(n-1)}} \quad (6.A-13)$$

where $V_{\text{median}}(h_j)$ is the generic median profile developed in Section 6.2.2. In Equation 6.A-12, the site-specific conditional mean log-velocity profile is a weighted sum of the generic profile and the average profile obtained from site-specific measurements (this is the common phenomenon of regression to the mean). The weight given to the site-specific average is high if n is large or ρ_s is high (i.e., if the site-specific sample is large or there is little variation among the site-specific profiles). Similarly, the site-specific conditional variability is low if n is large or ρ_s is high.

No procedure has been developed for the site-specific estimation of the serial correlation coefficient between layer log-velocities (Equation 6.A-5). The value obtained for the generic model ($\rho=0.577$) can be used as a first approximation. The quantities obtained in Equations 6.A-12 and 6.A-13, together with this value of ρ , can be used to generate artificial layer velocities by applying Equations 6.A-4 and 6.A-5 (using site-specific parameters in Equation 6.A-4).

No procedure has been developed for the conditional simulation of layering. The engineer must examine the layering (or stratigraphy) in the measured velocity profiles and select among the three approaches described below. If the layering is fairly similar across profiles, the engineer can use the stratigraphy from one of the measured sites as representative and adopt this as the stratigraphy for the artificial profiles. If the layering is less similar among profiles, the engineer can draw at random from the existing profiles. If this is not desirable, the engineer can use the generic Poisson layering model in Section 6.A.3. If the second or third options above are used, the site-specific median profile obtained in Section 6.A.12 must be smoothed to remove jumps associated with the stratigraphy of the measured site-specific profiles.

6.A.4.3 Examples of Modifications to the Model

This section describes how the model presented above can be modified to account for special situations. This is by no means an exhaustive list. The engineer's judgment and knowledge of the geology of site, coupled with the understanding of the limitations of the data, should guide these modifications.

If the plant site has a fairly uniform stratigraphy with a significant dip angle (or with some locations situated on eroded or excavated ground), the site-specific model should use a vertical frame of reference based on the stratigraphy rather than on the ground surface.

If the plot of observed standard deviation of log-velocity as a function of depth shows two or three distinct depth ranges with very different standard deviations (differing by factors of two or more), the assumption of constant $\sigma_{\ln(\text{Vel.})}$ with depth is not valid. In such case, these depth ranges should be analyzed separately.

If different geographic locations within the site have profiles with very distinct velocities, the profiles should be separated into clusters and analyzed separately. This situation, and the situation discussed above, have the disadvantage of smaller sample sizes and may require the collection of additional data.

An unlikely situation mentioned earlier occurs if the calculated site-specific standard deviation (σ_s) is larger than the generic standard deviation (0.39). This may be caused by the situation described above and may be resolved by separating the profiles into clusters. If this is not the case, the model developed here breaks down because a negative site correlation is counter-intuitive and because the covariance matrix may not be positive semi-definite. One may use the generic model (implicitly assuming that the site-specific data provide no useful information) or refine the model used here so that it distinguishes between the (unknown) true correlation ρ_s and its estimate (i.e., the quantity calculated in Equation 6.A-10).

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Table 6.A.1

Soil Profiles Considered for this Analysis

Profile Number	Profile Depth (ft)	Rock Depth (ft)	Depth (ft)	Profile Name
Shallower than Category I (<10 ft)				
246	52.5	1.6	1.6	Page Mill
444	98.4	3.3	3.3	APEEL 10-Skyline
9	90.9	3.9	3.9	Cholome 11-Temblor II
20	95.1	3.9	3.9	Vista Point
21	65.6	3.9	3.9	North Peak
138	91.9	3.9	3.9	Castaic Old Ridge Route
213	98.4	3.9	3.9	Pulgas Tunnel
366	45.0	5.0	5.0	Islais Creek-Hole 15
140	67.9	7.2	7.2	Warm Springs Camp
203	98.4	7.2	7.2	Mission Peak
256	36.7	7.5	7.5	Peninsula Country Club
14	98.4	8.2	8.2	El Granada
15	95.1	8.2	8.2	Black Mountain
16	101.7	8.2	8.2	Digges Canyon
17	91.9	8.2	8.2	Pise Lookout
19	95.8	8.2	8.2	Spring Valley Ridge
136	97.1	8.2	8.2	Edmonston Pumping Plant
142	72.8	8.2	8.2	Allen Ranch
211	95.1	8.2	8.2	Foothills Park
259	98.4	8.2	8.2	Crest Road
405	91.9	8.2	8.2	Sky Terrace
443	98.4	8.2	8.2	APEEL 7-Pulgas Water
11	98.4	8.5	8.5	Purisma
Category I (10-30 ft)				
206	98.4	10.5	10.5	Cal State Hayward
445	98.4	10.5	10.5	APEEL 3E-Hayward CSUH
209	100.1	16.1	16.1	Corte Madera
404	65.6	18.0	18.0	Diamond Bar
217	98.4	18.4	18.4	Morrison Canyon
139	98.4	19.0	19.0	Elizabeth Lake Fire Station
408	93.5	20.0	20.0	Hilltop House
409	95.1	20.0	20.0	Cedar Hills Nursery
425	82.0	23.0	23.0	Verdugo Park
234	44.9	26.2	26.2	UCSC Lick Observ-Agbab DH
391	30.0		30.0	Port of Stockton-hole 10
Category II (30-80 ft)				
214	67.6	33.8	33.8	Frenchman's Creek
446	98.4	41.0	41.0	Gilroy Gaviola College
208	42.0		42.0	Oliver's Salt Works
387	44.7		44.7	Richmond Dump-GT-3
382	45.0		45.0	Port of Stockton-hole 17
383	48.0		48.0	Port of Stockton-hole 5

Table 6.A.1 (continued)

Soil Profiles Considered for this Analysis

Profile Number	Profile Depth (ft)	Rock Depth (ft)	Depth (ft)	Profile Name
114	78.7	49.2	49.2	Castaic Dam
416	49.2		49.2	Compton Airport
6	182.1	50.2	50.2	Cholome 5-Cockrum's garage
90	50.2		50.2	Richmond Landfill-Hole GT1
111	82.0	57.4	57.4	Pearblossom Pump Plant
12	95.1	58.1	58.1	Cozzolino's
414	59.4		59.4	Lakewood Golf Course
216	98.4	62.0	62.0	Vista Grande
396	95.1	65.6	65.6	Shellmaker Island
421	65.6		65.6	Katella School
107	67.9		67.9	Leona Valley F.S.
247	90.9	68.9	68.9	Southern Pacific
413	69.6		69.6	Terminal Island
378	70.0		70.0	Olmsted Dam-KY side
244	70.5		70.5	Chain of Lakes
370	75.0		75.0	Islais Creek-Hole 21
145	78.7		78.7	CIT Athenaeum
215	98.4	78.7	78.7	Half Moon Bay Terrace
395	78.7		78.7	Burbank Fire Station
397	78.7		78.7	Cypress College
155	79.1		79.1	Calipatria Fire Station

Category III (80-180 ft)

199	80.1	80.1	Lowry Road
109	82.0	82.0	Littlerock Post Office
197	82.0	82.0	Blackberry Farm
202	82.0	82.0	Palo Alto Square
261	82.0	82.0	Santa Clara County Fairgrounds
401	82.0	82.0	Alta Vista Park
149	85.3	85.3	Santa Barbara Cnty Courthouse
402	85.3	85.3	Seal Beach Weapons Station
412	85.3	85.3	1st Lutheran Church
207	86.9	86.9	Miller School
251	87.3	87.3	Hillview
8	87.6	87.6	Cholome 12-Shandon Vineyards
106	87.6	87.6	Lake Hughes F.S.
265	87.6	87.6	Merrit Sand
137	87.9	87.9	OSO Pumping Plant
148	88.6	88.6	Palos Verdes
263	88.6	88.6	Carmichael School
427	88.6	88.6	Upper Van Norman Dam
36	90.2	90.2	El Centro Array #12
93	90.2	90.2	Alhambra
95	90.2	90.2	LA - Olive
102	90.2	90.2	Olive View
110	90.2	90.2	Llano South
200	90.2	90.2	Hayward Sink

Table 6.A.1 (continued)

Soil Profiles Considered for this Analysis

Profile Number	Profile Depth (ft)	Rock Depth (ft)	Depth (ft)	Profile Name
255	90.2		90.2	Westmoor
260	90.2		90.2	Audubon School
156	90.9		90.9	Salton Sea Wildlife Ref.
18	91.9		91.9	Pulgas Water Temple
254	91.9		91.9	Skyline
399	91.9		91.9	Sierra Linda School
407	91.9		91.9	Sylmar Park
411	91.9		91.9	Magnolia
415	91.9		91.9	F.S. 105
423	91.9		91.9	Rowland Heights
13	92.8		92.8	MaryKnoll
435	93.5		93.5	Ventura Co. Airport
94	94.2		94.2	Vernon
154	94.2		94.2	Parachute Test Site
258	94.5		94.5	KGEI
101	95.1		95.1	Devonshire Police Station
201	95.1		95.1	Parkway Towers
392	95.1		95.1	Camarillo State Hospital
398	95.1		95.1	Ventura Pistol Range
406	95.1		95.1	Sylmar Nursery
410	95.1		95.1	Cal State Northridge
418	95.1		95.1	Holiday Inn
419	95.1		95.1	Veterans Hospital
422	95.1		95.1	St. Jude's Hospital
424	95.1		95.1	El Monte Courthouse
428	95.1		95.1	Pacoima Memorial Hospital
430	95.1		95.1	Sylmar High School
431	95.1		95.1	Mission Hills P.O.
438	95.1		95.1	Bard Sanitarium
394	95.8		95.8	Westminster H.S.
198	96.5		96.5	John Muir School
400	96.8		96.8	San Miguel School
426	96.8		96.8	20th Century
242	97.1		97.1	Redwood City-Bridgeway Prk
196	97.4		97.4	Alviso Raceway
96	97.8		97.8	LA - Hill
243	97.8		97.8	Wind Mill
250	97.8		97.8	Quintara (Sunset District)
97	98.4		98.4	Hollywood Storage
98	98.4		98.4	Santa Monica-Wilshire
99	98.4		98.4	Tishman Airport Center
100	98.4		98.4	Hyperion
103	98.4		98.4	Mullholland Jr. H.S.
104	98.4		98.4	Camp Munz
108	98.4		98.4	Llano North
112	98.4		98.4	Holiday Inn Palmdale
113	98.4		98.4	Palmdale F.S.
135	98.4		98.4	Wheeler Ridge

Table 6.A.1 (continued)

Soil Profiles Considered for this Analysis

Profile Number	Profile Depth (ft)	Rock Depth (ft)	Depth (ft)	Profile Name
141	98.4		98.4	Wrightwood
143	98.4		98.4	Cedar Springs Dam
144	98.4		98.4	Colton SCE
153	98.4		98.4	Westmorland Fire Station
204	98.4		98.4	Montgomery School
210	98.4		98.4	Mitchell Park
218	98.4		98.4	Lake Merced Country Club
248	98.4		98.4	Avalon
264	98.4		98.4	Oak Avenue School
432	98.4		98.4	Eton School
434	98.4		98.4	Pacific Missile Test Center
436	98.4		98.4	Mandalay Beach
442	98.4		98.4	APEEL 9-Crystal Springs
447	98.4		98.4	APEEL 2E-John Muir School
450	98.4		98.4	Gilroy 4-San Ysidro School
452	98.4		98.4	APEEL 2-Redwood City
453	98.4		98.4	APEEL 1-Foster City
221	98.8		98.8	Capitola-Agbab DH
7	99.7		99.7	Cholome 8-Shandon Pump Sta
433	100.1		100.1	Camarillo State Hospital
437	100.1		100.1	Ventura Co. General Hospital
417	101.7		101.7	Compton Civic Center
151	109.9		109.9	Imperial Co. Serv. Bldg.
369	110.0		110.0	Islais Creek-Hole 19
371	110.0		110.0	Islais Creek-Hole 4
372	110.0		110.0	Islais Creek-Hole 5
81	114.5	114.5	114.5	Forgaria, Italy
180	115.2	115.0	115.0	Beaver Valley
368	115.0		115.0	Islais Creek-Hole 16
89	147.6	118.1	118.1	Richmond Field Station
385	120.0		120.0	Richmond Dump-GT-1
152	126.0		126.0	Brawley Airport
393	128.0		128.0	Marina Del Rey
384	130.0		130.0	Richmond Dump-averaged
386	130.0		130.0	Richmond Dump-GT-2
91	134.8		134.8	Richmond Landfil-Hole GT 1&2
105	150.9		150.9	Rosamond Dry Lake
420	150.9		150.9	Newport Beach SDP 2
377	153.0		153.0	Olmsted Dam-KY side
219	164.0		164.0	Agnews State Hosp-Agbab DH
381	173.0		173.0	Olmsted Dam-IL side general

Category IV (180-400 ft)

23	180.4		180.4	Bonds Corner
31	189.3		189.3	El Centro Array #7
10	196.9		196.9	Lincoln School
83	196.9		196.9	Lotung LSST

Table 6.A.1 (continued)

Soil Profiles Considered for this Analysis

Profile Number	Profile Depth (ft)	Rock Depth (ft)	Depth (ft)	Profile Name
225	196.9		196.9	Menlo Park VA-Agbab DH
449	196.9		196.9	Gilroy 3-Sewage Treat Plant
373	200.0		200.0	LLNL-NE Corner EW-Poleriz
374	200.0		200.0	LLNL-NE Corner NS-Poleriz
35	205.4		205.4	El Centro Array #11
367	210.0		210.0	Embarcadero B1
27	222.4		222.4	El Centro Array #3
29	225.4		225.4	El Centro Array #5
30	229.7		229.7	El Centro Array #6
34	233.3		233.3	El Centro Array #10
37	235.6		235.6	El Centro Array #13
227	236.2		236.2	Fremont Mission SJ-Agbab DH
232	236.2		236.2	Richmond City Hall-Agbab DH
32	238.2		238.2	El Centro Array #8
150	239.8		239.8	El Centro Differential G.M.A.
28	242.1		242.1	El Centro Array #4
25	246.1		246.1	Holtville Post Office
26	247.4		247.4	El Centro Array #2
24	248.0		248.0	Calexico Fire Station
310	259.1		259.1	Savannah River NPR M12A-17
82	284.8	284.8	284.8	Southern Pacific Bldg
351	328.1		328.1	Treasure Isl.-EPRI Hole USGS
451	344.5		344.5	Hollister City Hall
235	374.0		374.0	Oakland 1700 Webster
38	390.1		390.1	El Centro Differential G.M.A.
189	393.7		393.7	Gilroy 2 USGS hole
40	400.0		400.0	Savannah River P-25

Category V (400-1000 ft)

193	483.9	434.7	434.7	Palo Alto Veterans hospital
191	475.7	437.0	437.0	S.F. International Airport SFO
388	500.0	437.0	437.0	SF Airport-Redpath
39	450.0		450.0	Savannah River SSW-1
186	459.3		459.3	Alameda Naval Air Station
363	511.8	467.5	467.5	Oakland Bay Bridge, interprtd
375	480.0		480.0	Oakland Harbor-Redpath
190	551.2	492.1	492.1	Oakland Outer Harbor Wharf
289	549.9		549.9	Gilroy 2 EPRI hole-Redp interp
364	560.0		560.0	Cypress Structure hole B-2
365	570.0		570.0	Cypress Structure hole B-8
389	623.4		623.4	Vineyard Canyon
33	801.8		801.8	El Centro Array #9

PROFILES CONSIDERED
IN THIS STUDY

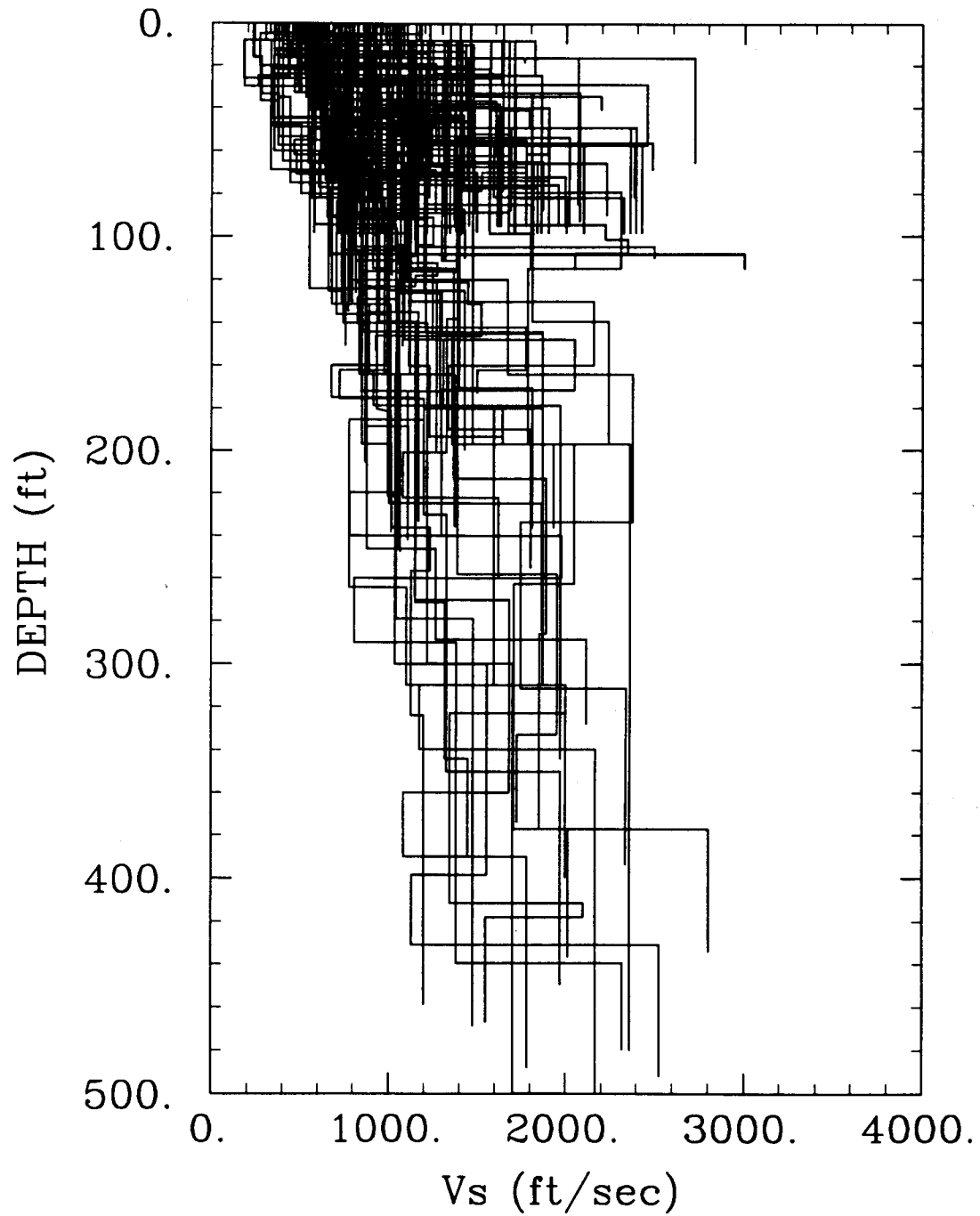


Figure 6.A-1
Soil profiles selected for analysis.

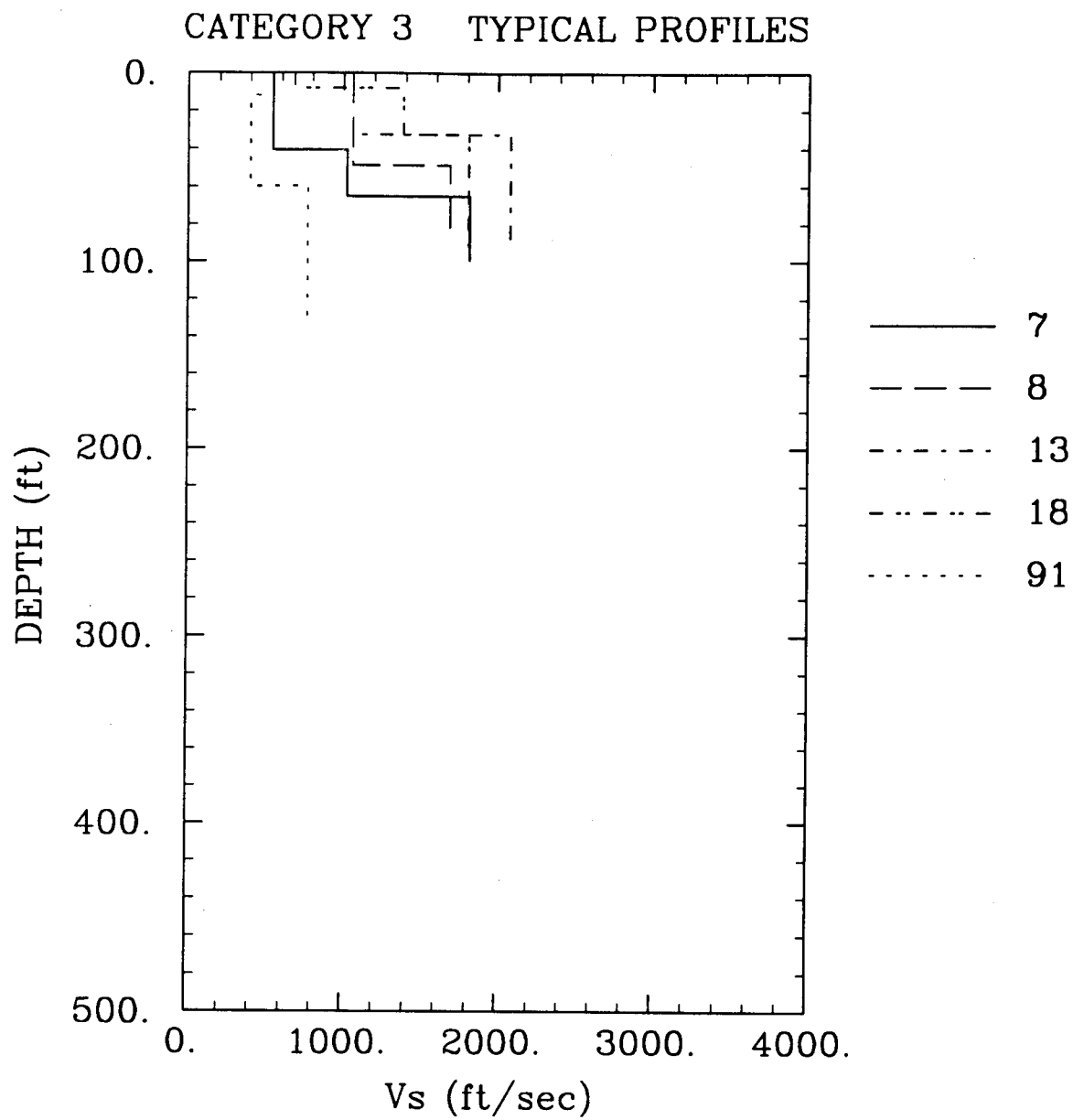


Figure 6.A-2
Five category III profiles. The numbers correspond to profile numbers in the EPRI soil database.

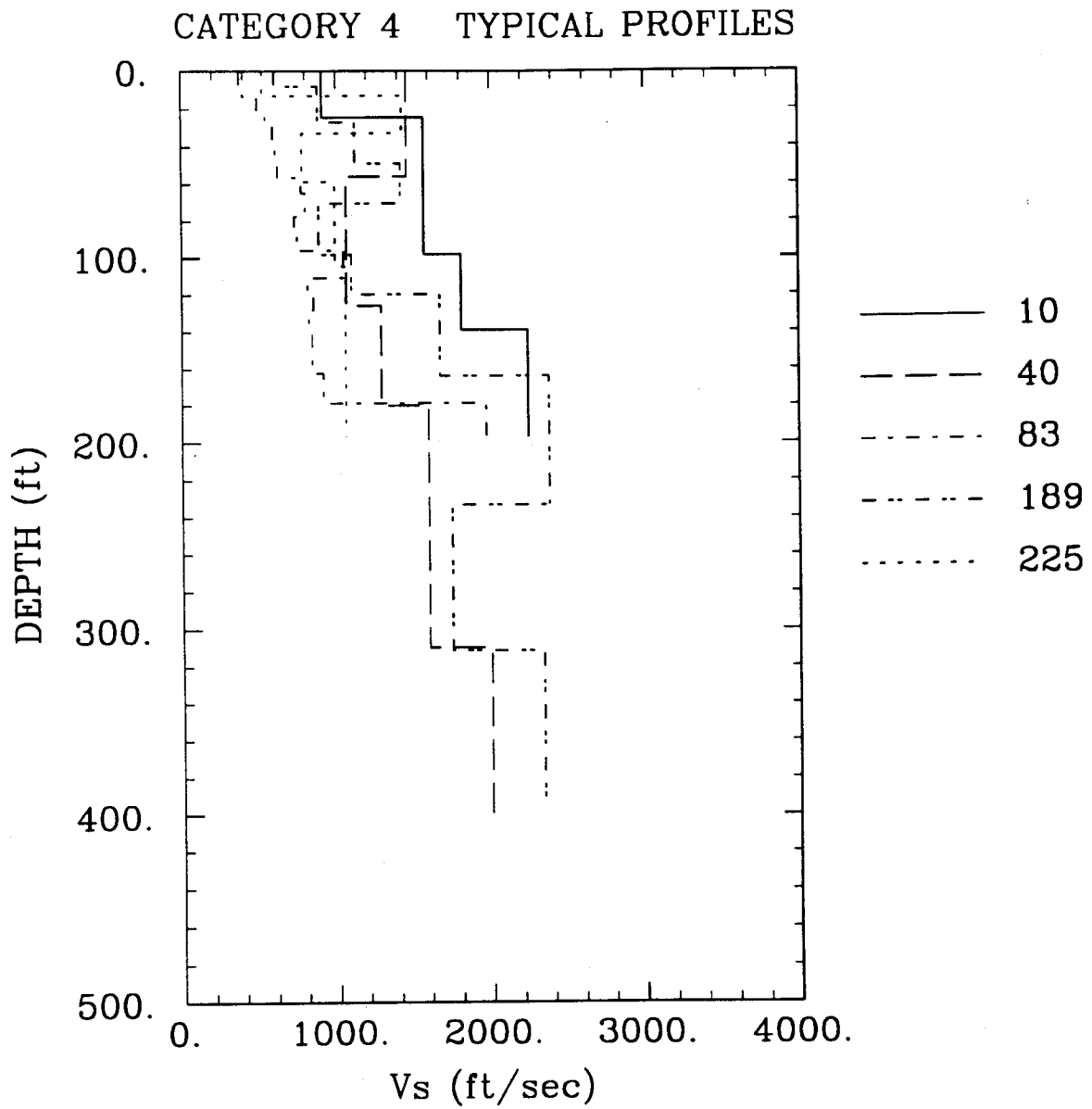


Figure 6.A-3

Five category IV profiles. The numbers correspond to profile numbers in the EPRI soil database.

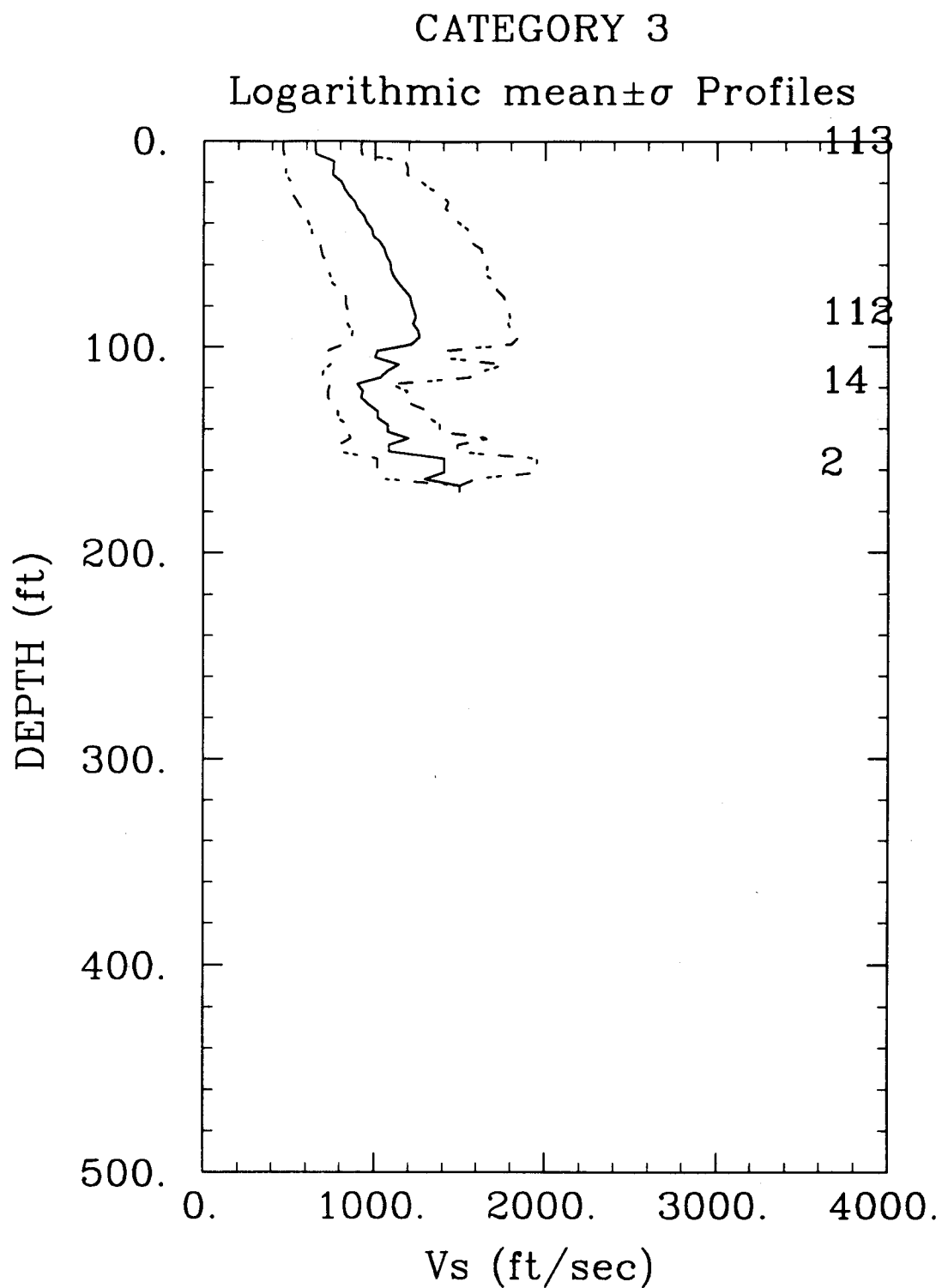


Figure 6.A-4

Logarithmic mean $\pm$ standard deviation for category III profiles. The reduction in the mean near 100 ft. is caused by the termination of several profiles with high velocities.

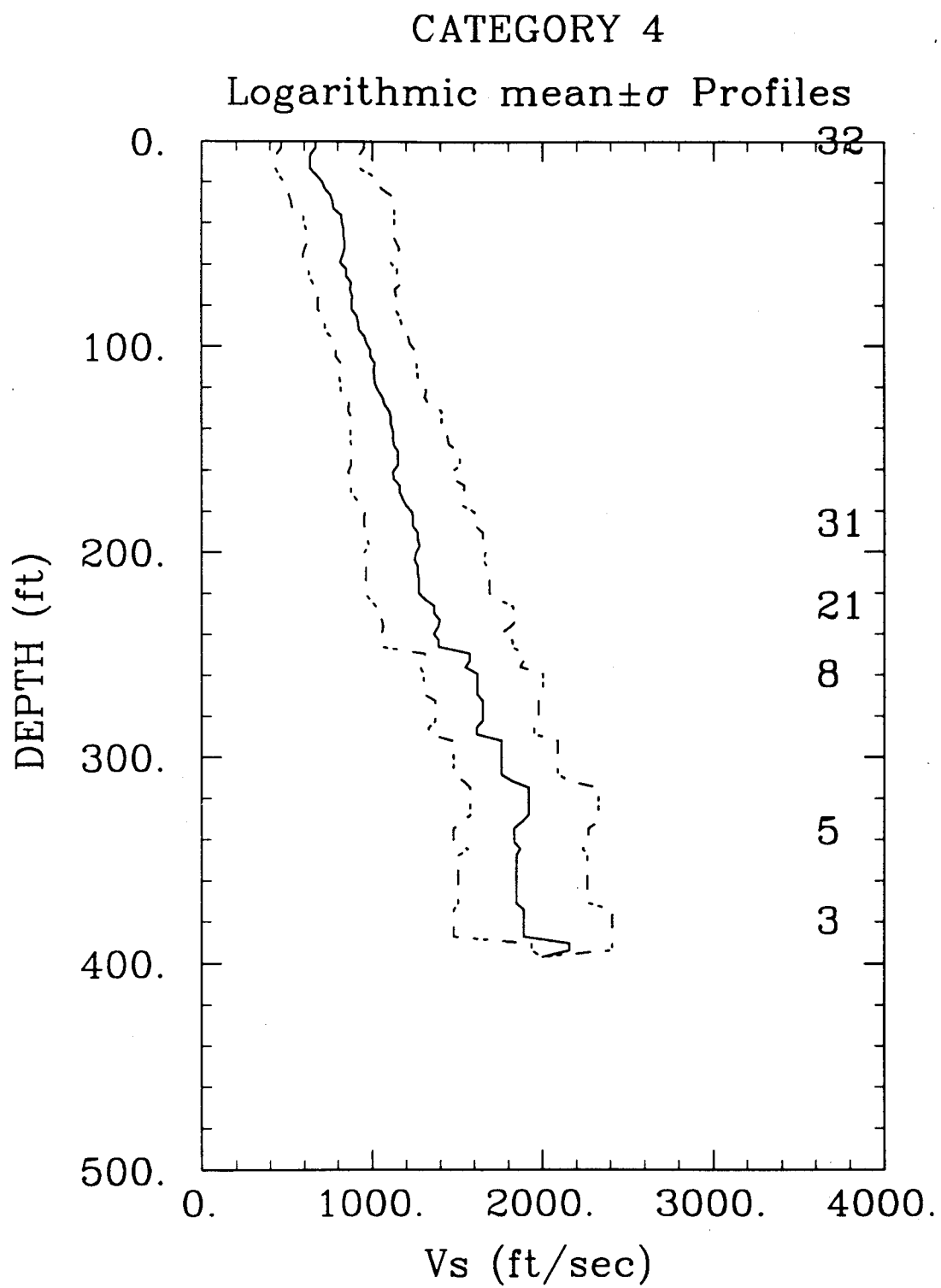


Figure 6.A-5
Logarithmic mean $\pm$ standard deviation for category IV profiles.

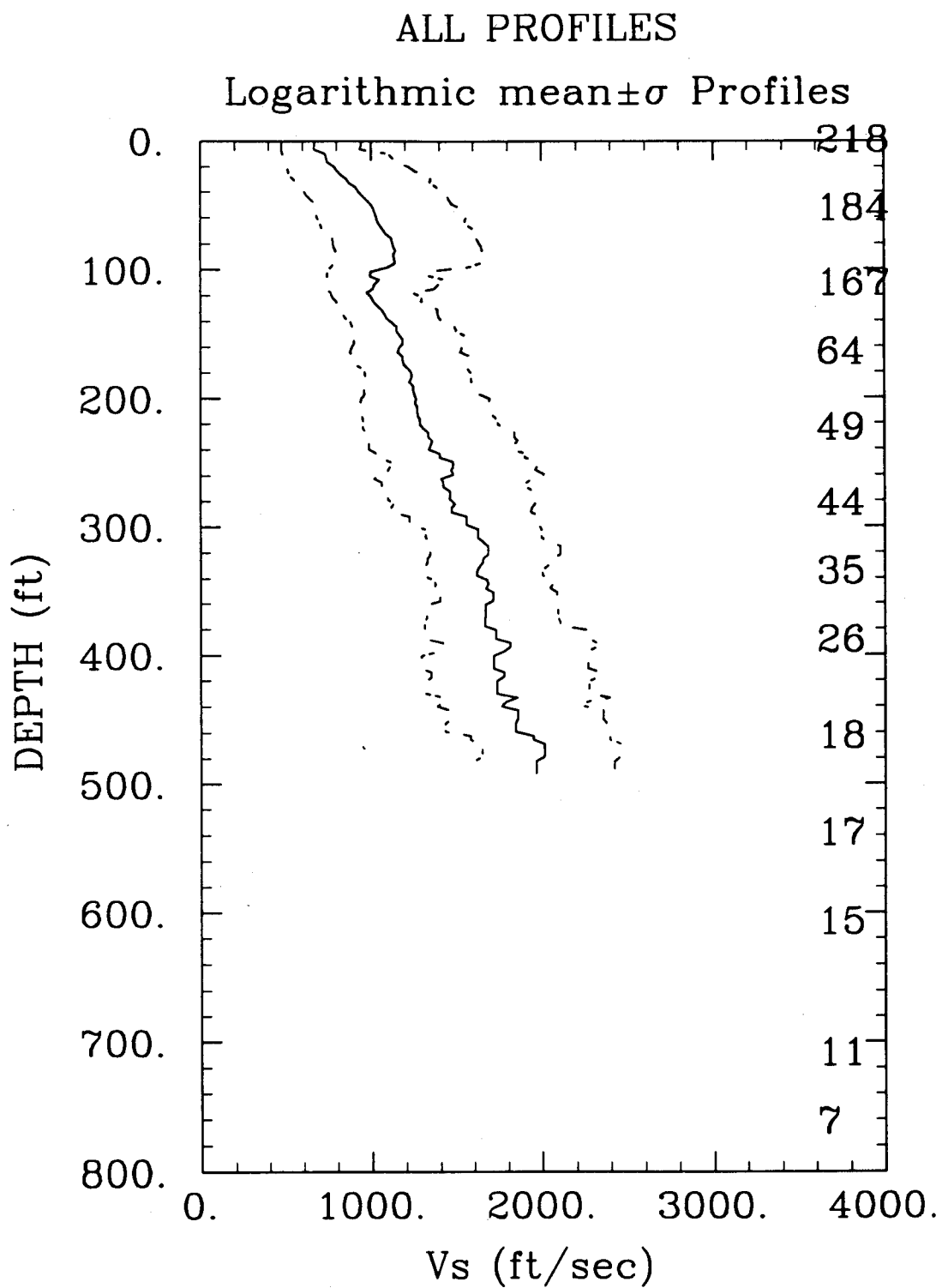


Figure 6.A-6
Logarithmic mean $\pm$ standard deviation for all profiles used in analysis.

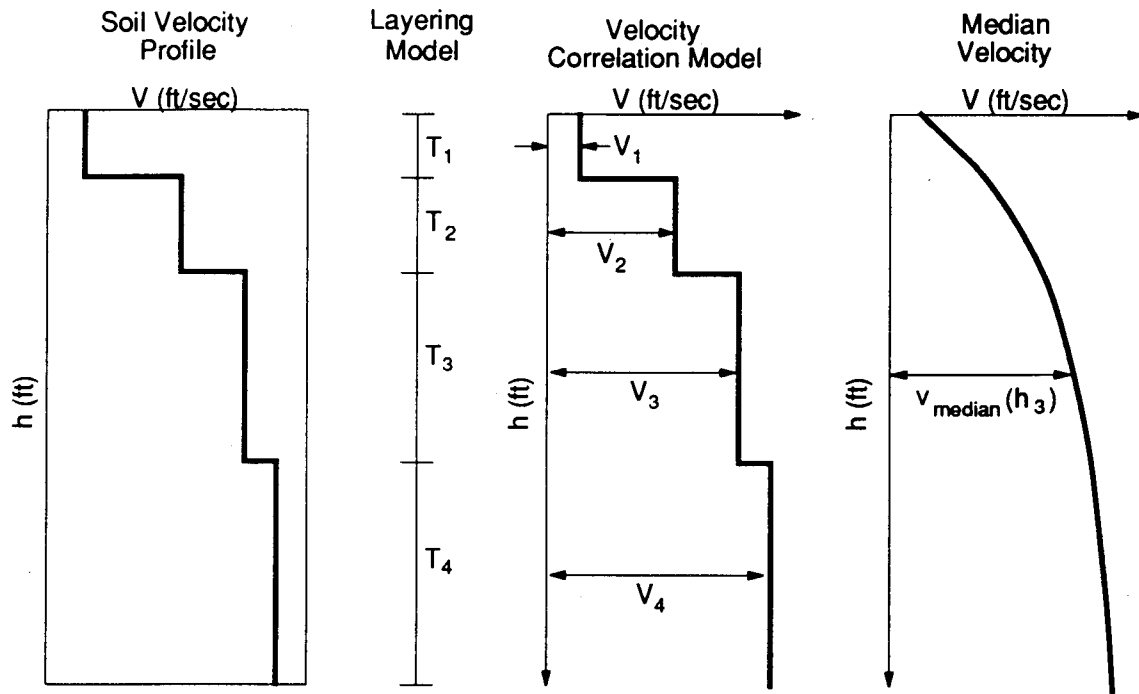


Figure 6.A-7
Elements of the probabilistic model for soil velocity profiles.

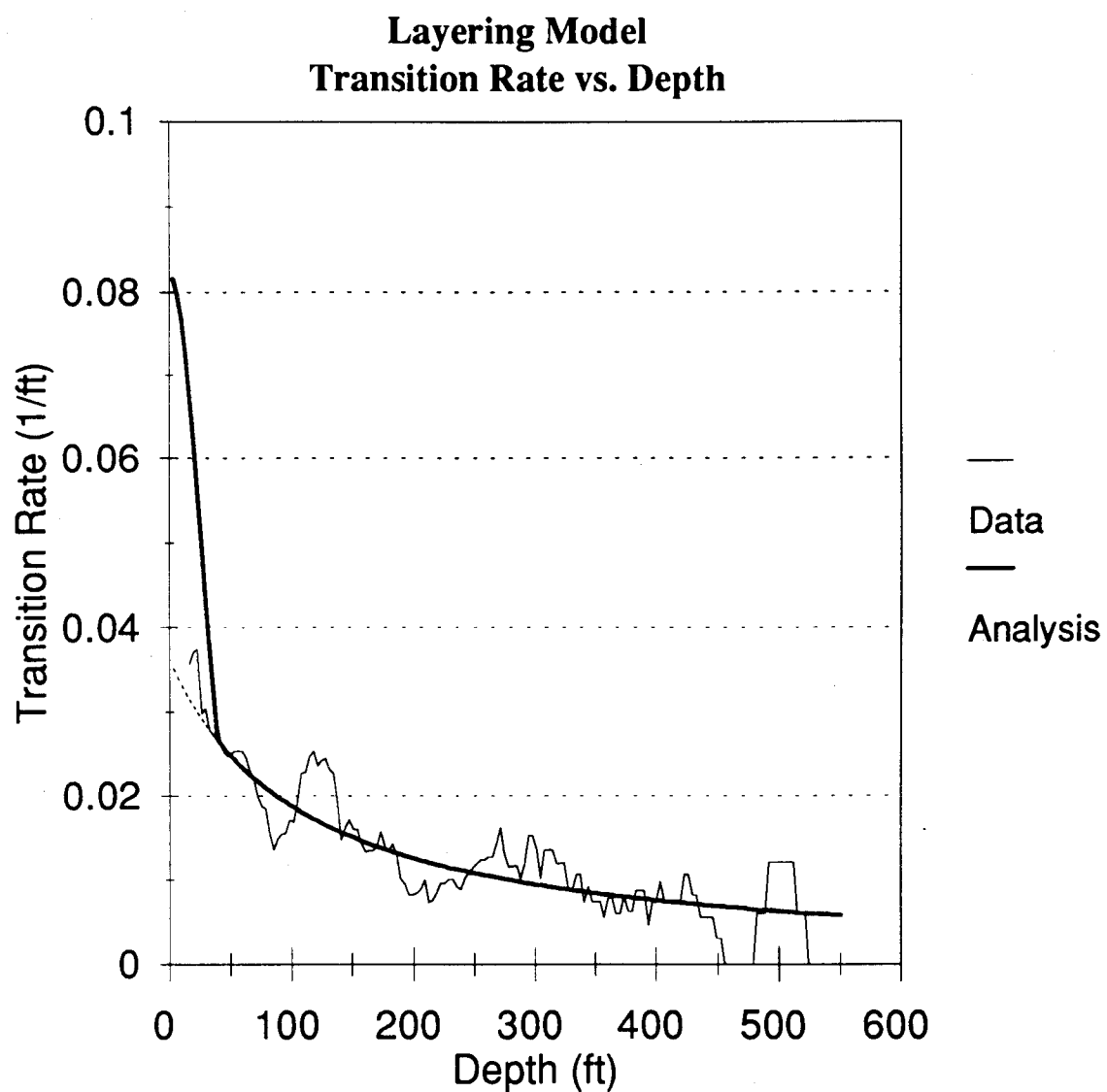


Figure 6.A-8

Model for the rate of layer boundaries (or transition rate). Jagged line: Observations (running averaged); Thin smooth curve: fitted model; heavy smooth curve: modified model.

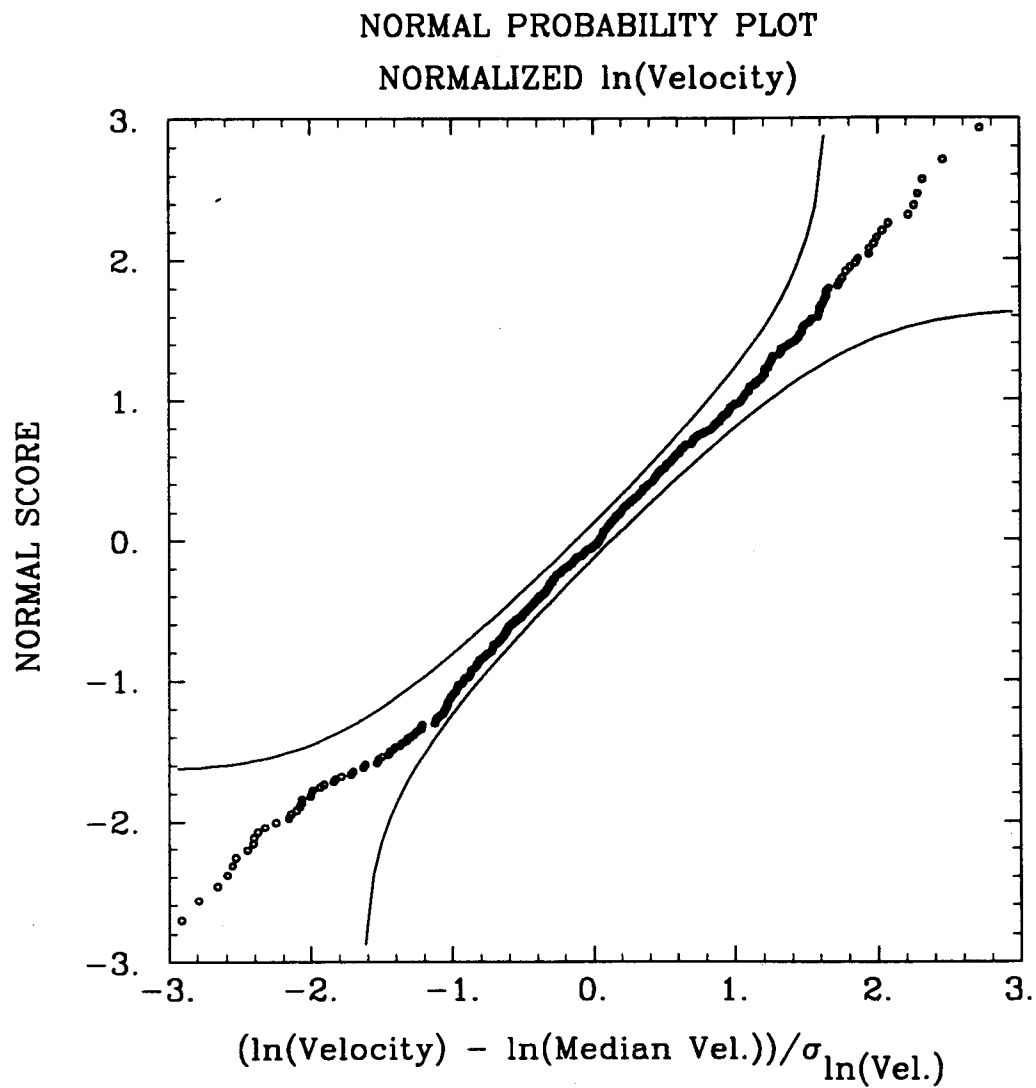


Figure 6.A-9

Log-normal probability plot of shear-wave velocities. Smooth curves: 10% Kolmogorov-Smirnov bounds.

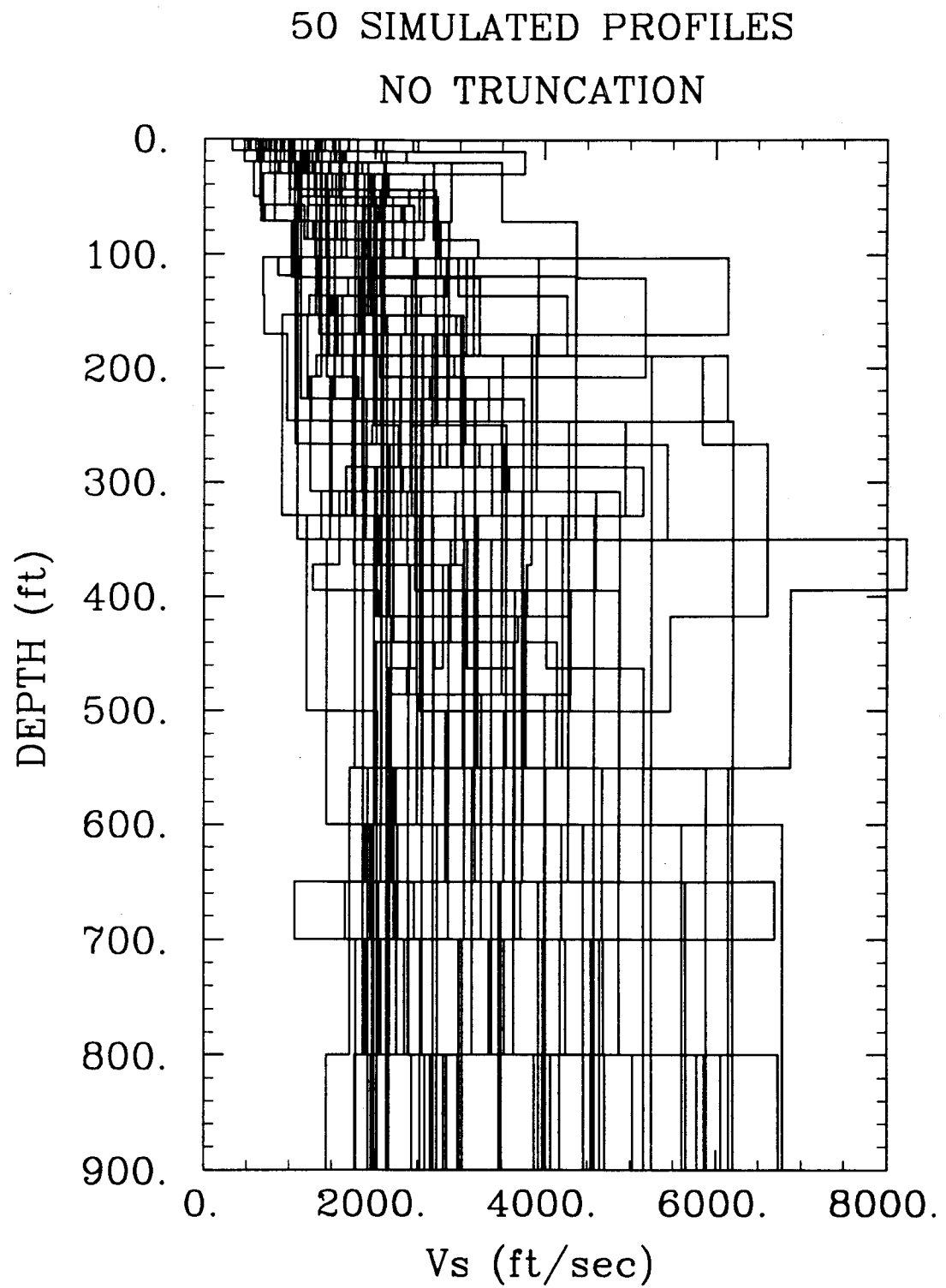


Figure 6.A-10
Fifty simulated profiles; model with no truncation.

50 SIMULATED PROFILES
TRUNCATED RESIDUALS AND VELOCITIES

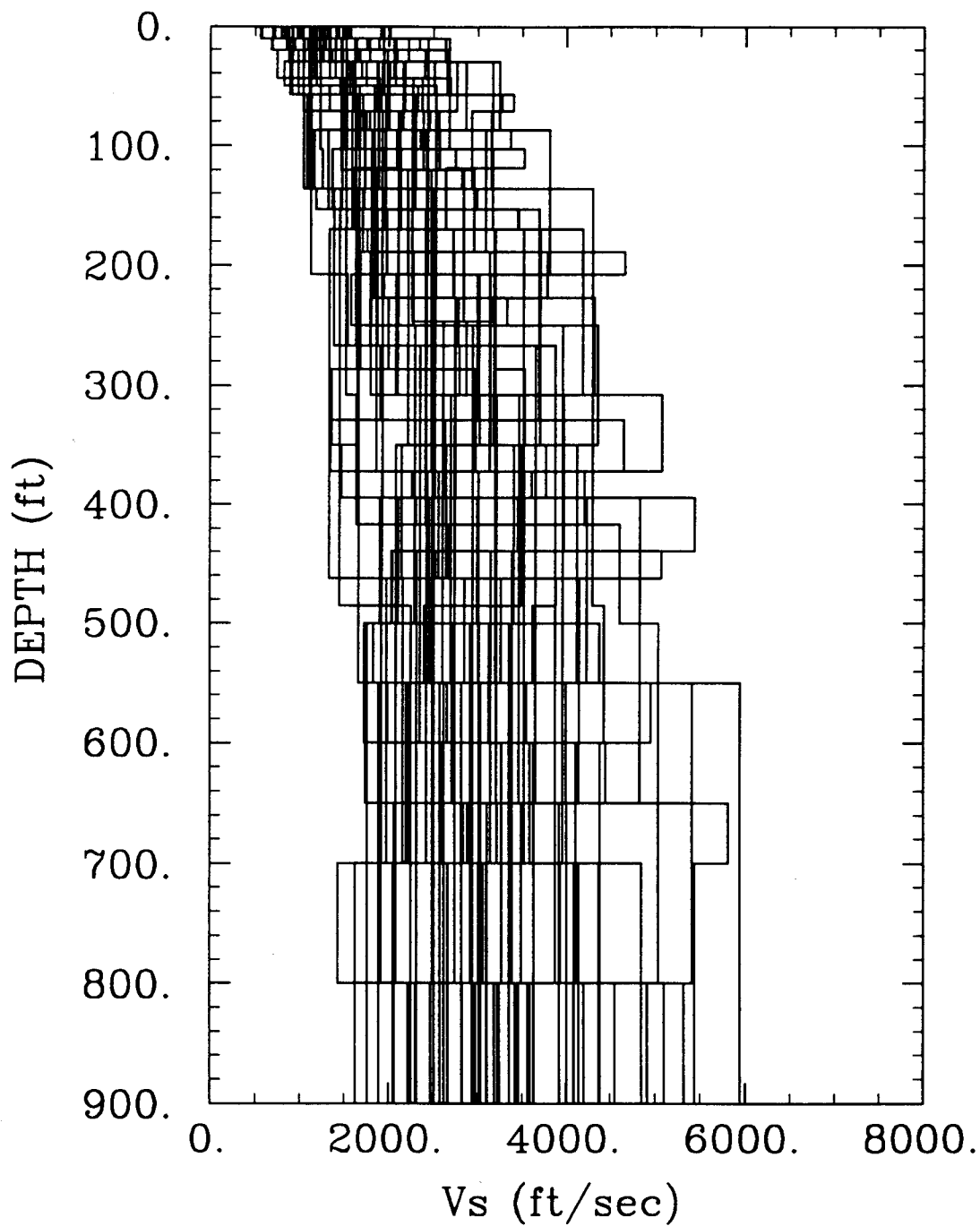


Figure 6.A-11
Fifty simulated profiles; model with truncation.

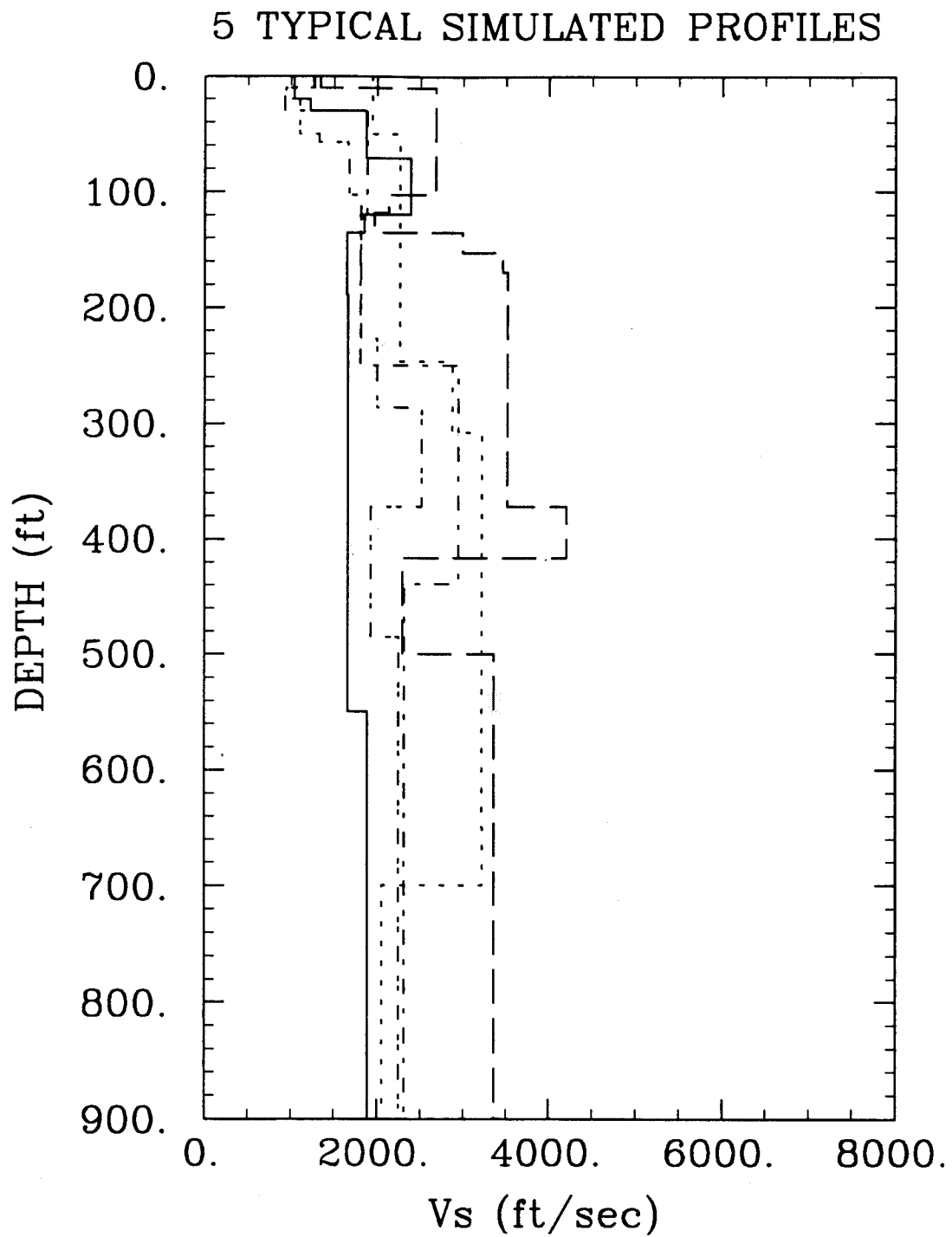


Figure 6.A-12
Five typical simulated velocity profiles.

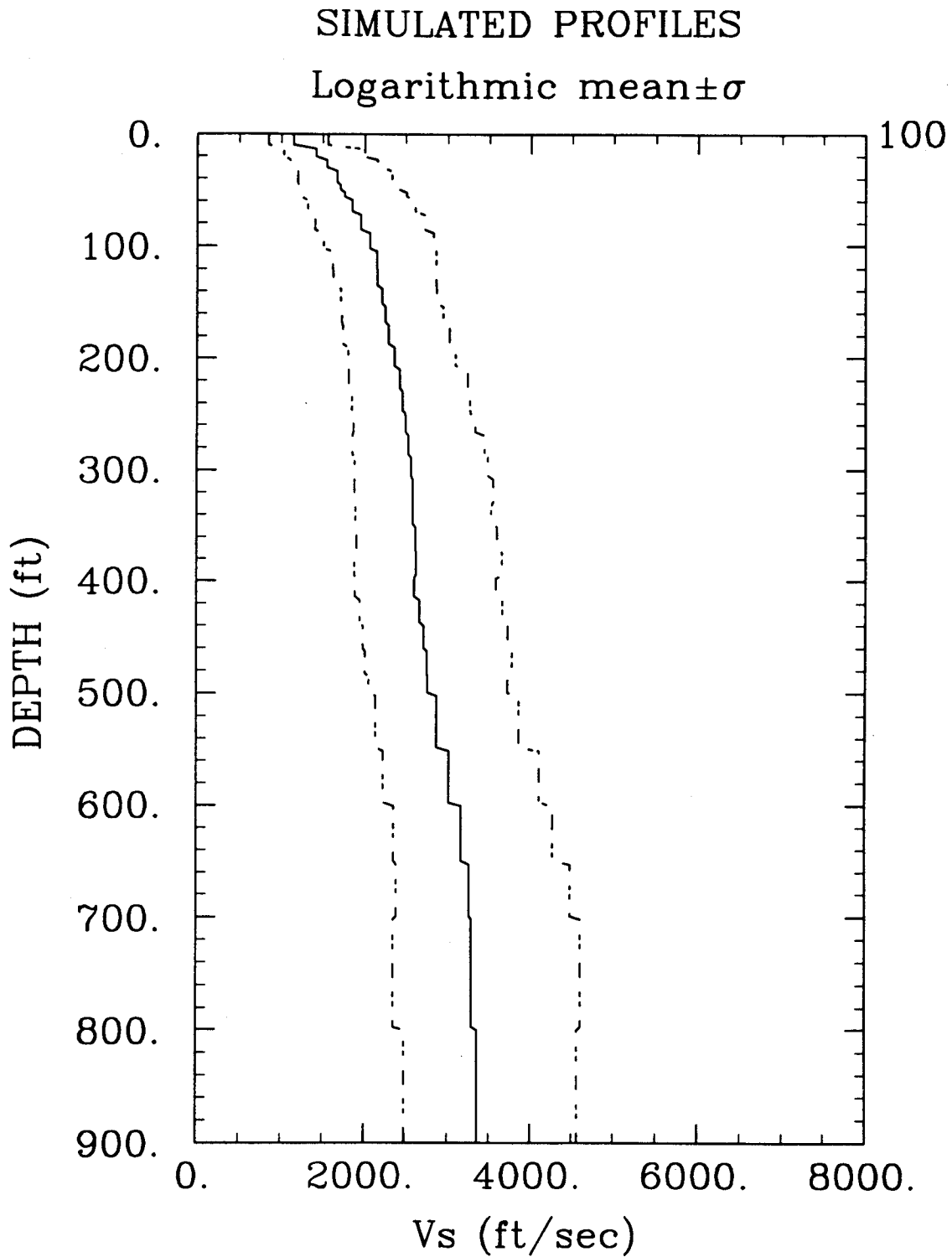


Figure 6.A-13
Logarithmic mean $\pm$ standard deviation of 100 simulated velocity profiles.

APPENDIX 6.B

VALIDATION AND COMPARISON OF ONE-DIMENSIONAL SITE RESPONSE METHODOLOGIES

6.B.1 Introduction

The effects of a soil column upon strong ground motion have been well documented and studied analytically for many years. Wood (1908) and Reid (1910), using apparent intensity of shaking and distribution of damage from the 1906 San Francisco earthquake, gave evidence that the severity of shaking can be substantially affected by the local geology and soil conditions. Gutenberg (1957) developed amplification factors representing different site geology by examining recordings of microseisms and earthquakes from instruments located on various types of site geology. More recently, Wiggins (1964), Idriss and Seed (1968), Seed et al. (1969), Borchardt and Gibbs (1976), Joyner et al. (1976), Berrill (1977), Duke and Mal (1978), Chin and Aki (1991), Darragh and Shakal (1991), Silva (1991), Hartzell (1992), Silva and Stark (1992), and Su et al. (1992) have shown that during small and large earthquakes, the surface soil motion can differ in significant and predictable ways from that on adjacent rock outcrops. Other investigators have utilized explosion data either independently or in conjunction with earthquake data to examine site response characteristics (Murphy et al., 1971; Hays et al., 1979; Rogers et al., 1984). Recent work using horizontal as well as vertical arrays of instruments have demonstrated the general consistency of the site response for seismic events of different sizes, distances, and azimuths (Tucker and King, 1984; Benites et al., 1985; Hutchins and Wu, 1990; Chin and Aki, 1991; Borchardt and Glassmoyer, 1992; Field et al., 1992; Su et al., 1992). Results of these and other studies have demonstrated, in a general sense, the adequacy of assuming plane-wave propagation in modeling one-dimensional site response for engineering purposes.

6.B.1.1 Wave Propagation Modeling

In addition to the wave propagation model used in site response analyses, dynamic material properties such as shear modulus and material damping control the amplitude and frequency content of computed motions. It is well known from laboratory testing that soils (and indeed soft rocks) exhibit pronounced nonlinear behavior under shear loading conditions. Shear modulus decreases with increasing strain with an accompanying increase in material damping (decrease in Q_s) (Drnevich et al., 1966; Seed and Idriss, 1970; Hardin and Drnevich, 1972; Seed et al., 1984). If this observation is applicable to in situ soil properties subject to earthquake loading, then site response calculations must accommodate these strain dependencies as nonlinear constitutive relations.

6.B.1.1.1 Nonlinear Model. In general two approaches are conventionally used to model cyclic soil response: nonlinear and equivalent-linear. The two main components of nonlinear analyses are a solution (integration) scheme for the wave equation and a nonlinear soil model. Nonlinear wave propagation techniques utilize finite element (Day, 1979) schemes, implicit or explicit finite difference (Joyner and Chen, 1975; Martin, 1975) schemes, or the method of characteristics (Streeter et al., 1974). One approach incorporates a general nonlinear soil model into an explicit finite difference approach which retains second order terms (Valera et al., 1978; Moriwaki et al., 1981).

Nonlinear soil models which have been primarily developed from laboratory test results and utilized in dynamic analyses, include the Ramberg-Osgood model (Faccioli et al., 1973; Streeter et al., 1974), an elasto-plastic model (Richart, 1975), Iwan-type model (Joyner and Chen, 1975; Taylor and Larkin, 1978;

Valera et al., 1978), the hyperbolic model (Hardin and Drnevich, 1972), the endochronic model (Day, 1979), and the Davidenkov model (Martin, 1975; Pyke, 1979).

Each of the nonlinear models mentioned has certain limitations and advantages in describing the response of soils to the type of loading produced by seismic disturbances. An effort has been made in some models to predict permanent deformations, while others have included pore pressure build-up and dissipation. Strain dependency of material properties from laboratory data is universally observed. It is reproducible and becomes significant for high levels of earthquake loading, i.e., strains $> 10^{-2}\%$.

6.B.1.1.2 Equivalent-Linear Model. The equivalent-linear approach, in its present form, was introduced by Seed and Idriss (1970). This scheme is a particular application of the general equivalent-linear theory introduced by Iwan (1967). Basically, the approach is to approximate a second order nonlinear equation, over a limited range of its variables, by a linear equation. This was done in an ad hoc manner for ground response modeling by defining an effective strain which is assumed to exist for the duration of the excitation. This figure is usually taken as 65% of the peak time-domain strain calculated at the midpoint of each layer, using a linear analysis. Moduli and damping curves are then used to define new parameters for each layer. The linear response calculation is repeated, new effective strains evaluated, and iterations performed until the changes in parameters are below some tolerance level. Generally a few iterations are sufficient to achieve a strain-compatible linear solution. This stepwise analysis procedure was formalized into a one-dimensional, vertically propagating shear-wave code termed SHAKE (Schnabel et al., 1972). Subsequently, this code has become easily the most widely used analysis package to perform one-dimensional site response calculations.

The advantages of the equivalent-linear approach are that the mathematical simplicity of linear analysis is preserved and the determination of nonlinear parameters is avoided. A truly nonlinear approach requires the specification of the shapes of hysteresis curves and their cyclic dependencies. In the equivalent-linear methodology the soil data are utilized directly and, because at each iteration the problem is linear and material properties are frequency independent, the damping is rate independent and hysteresis loops close.

A significant advantage of the equivalent-linear formulation is the preservation of the superposition principle. For linear systems, this principle permits, among other things, spectral decomposition and frequency-domain solutions. One can then appeal to the elegant propagator matrix solution scheme (Haskell, 1960; Schnabel et al., 1972; Silva, 1976) for very efficient frequency domain solutions of the wave equation. The superposition principle then permits a spectral recomposition of the wavefields (sum over frequencies) through an inverse Fourier or Laplace transform. A non-subtle result of this is that the deconvolution process, that of propagating the control motion down rather than up, results in a unique solution. That is, for a given motion at the surface, within an equivalent linear framework there is only one input motion (solution). In reality of course, if the soils are behaving in a nonlinear fashion and have degraded, many different input motions at the base of the soil could have resulted in a similar surface response.

The main disadvantage of the equivalent-linear approach is that it gives poor predictions of large strains and therefore cannot model soil deformation or failure. Additionally, in response calculations, there is always some difference between an equivalent-linear result and that using a fully nonlinear analysis particularly for soft soils or soils that liquefy.

6.B.1.2 Site Response Issues

The three fundamental issues in site response analyses are then the adequacy of the vertically propagating shear-wave model, the determination of in situ dynamic soil properties, and the suitability of the approximate equivalent-linear method compared to fully nonlinear schemes. It is the purpose of this appendix to address these issues by comparing site response analyses using equivalent-linear and nonlinear analyses to high levels of recorded motions at three well characterized reference sites which represent a wide range in soil conditions: Gilroy 2, Treasure Island, and Lotung, Taiwan. Additionally,

results of equivalent-linear and nonlinear analyses will be compared to each other for a hypothetical 120 ft deep clean sand site using control motions ranging from 0.50 to 1.25g.

6.B.2 Reference Sites

Three sites were chosen to provide a basis for an evaluation of the appropriateness of conventional one-dimensional site response analysis. Criteria for the site selection were based upon 1) proximity of soil site to nearby rock outcrop (within 2–3 km) or a vertical instrument array; 2) availability of recordings of both weak and strong seismic motions; 3) accessibility of site to drilling and testing; and 4) suitability of soil in terms of stiffness and particle size for obtaining undisturbed samples and performing dynamic soil tests; and 5) representation of a broad range in soil conditions and type. The three reference sites are Treasure Island with a nearby rock outcrop at Yerba Buena Island, Gilroy 2 with Gilroy 1 as a nearby rock outcrop, and Lotung, Taiwan with a vertical array of three component accelerometers.

6.B.2.1 Reference Site Geotechnical Models

A complete geotechnical and geophysical description of the reference sites is presented in Section 8 and its appendixes. The following discussion on the three sites is a summary presentation of the results of both the field and laboratory investigations.

6.B.2.1.1 Gilroy 2 Reference Site. A series of geophysical surveys were performed to measure shear- and compression-wave velocities at Gilroy 2 (Section 8.2). Figure 6.B-1 shows the base-case or best-estimate shear- and compression-wave velocity profiles for Gilroy 2. The profiles represent an average of the in-situ velocities shown in Figure 6.B-2. Velocity and unit weight values for the profiles are listed in Table 6.B-1. Tables 6.B-2 and 6.B-3 show the geologic log and soil sample properties respectively. At this site nine undisturbed samples were taken over the depth range of 10 to 420 ft. A complete discussion of the laboratory testing is given in Section 8.3.

The modulus reduction and damping curves resulting from the laboratory dynamic testing are shown in Figure 6.B-3 and are listed in Table 6.B-4. The two sets of curves are for depth ranges of 0–130 ft and beyond 130 ft. Due to the similarity of modulus ratio and damping data in the upper 130 ft in Table 6.B-4, average curves were developed for this depth range. In this case, the increased damping and lower modulus reduction at greater depths, which is counter to the effects of increasing confining pressure, are due to the presence of gravels below 130 ft (Table 6.B-2). Interestingly, the low-strain in situ damping measurements (Section 8.2) also reflect the increased damping (lower Q_s) within the gravels.

The geotechnical model of this reference site then reflects an assessment of a suite of in situ compression and shear-wave velocity and damping measurements as well as laboratory testing of undisturbed samples taken throughout the profile. The profile is generally stiff consisting of sands, clays, and soils near the surface with gravels being the predominant component of the materials from about 130 ft to near 400 ft. Weathered bedrock is reached at about 550 ft with a steep velocity gradient to about 650 ft (Figure 6.B-2).

6.B.2.1.2 Treasure Island Reference Site. As with the Gilroy 2 site, a suite of compression and shear-wave velocity and damping surveys were performed (Section 8.2). The base case velocity profiles are shown in Figure 6.B-4 and represent a best estimate average of the data shown in Figure 6.B-5. The velocities are listed on Table 6.B-5. The geologic log is shown in Table 6.B-6 and indicates a profile consisting of about 40 ft of sands (hydraulic fill) overlying Young Bay mud and then Old Bay clay with bedrock occurring at a depth of about 300 ft. Groundwater is located approximately 4 ft below the surface.

Since the hydraulic fill likely liquefied in places during the 1989 Loma Prieta earthquake, the generation and dissipation of excess pore pressure is an important consideration in nonlinear models of ground response. As a result, an estimate of the permeability of the hydraulic fill material of 5×10^{-4} ft/sec (Robert Pyke, personal communication) was used in the nonlinear analyses. Additionally an average

uncorrected SPT of 5–10 blows/ft, taken from other studies (Power et al., 1993), was used for the fill material in assessing some of the input parameters for the nonlinear analyses. Laboratory derived material properties are listed in Table 6.B-7 along with the depths at which the samples were taken.

Figure 6.B-6 shows the set of modulus reduction and damping curves which were based on the laboratory dynamic testing program (Section 8.3). The curves for the 0–44 ft depth range reflect the dynamic properties of the fill material. The deeper material is largely clays (bay mud and clay) with plasticity index values around 25%. The weaker strain dependencies on both the modulus reduction and damping reflect these properties. Table 6.B-8 list the modulus reduction and damping curves.

6.B.2.1.3 Lotung, Taiwan Reference Site. At this site, previous site investigations (Anderson and Tang, 1989) have defined the compression- and shear-wave velocity profiles. This information was used to develop base-case shear- and compression-wave velocities shown in Figure 6.B-7. Table 6.B-9 lists the velocity profiles and unit weights and the geologic log is shown in Table 6.B-10. The profile is largely uniform, reflected in the slowly varying shear-wave velocities with depth and consists of a silty to clayey sand. The profile and the additional boring to obtain undisturbed samples both extend to a depth of about 150 ft (47 m), the depth of the deepest accelerometer package. Groundwater is found at the ground surface.

The soil properties obtained from the laboratory testing are shown in Table 6.B-12. For the clayey silt samples, the PI is only about 7–8%, suggesting dynamic material properties closer to sands rather than highly plastic clays. This is reflected in the modulus reduction and damping curves shown in Figure 6.B-8. The single set of curves for all depths, based upon laboratory dynamic testing of the samples listed on Table 6.B-11 (Section 8.3), is consistent with the general uniformity shown in both the shear-wave velocity profile (Figure 6.B-7) and in material type (Table 6.B-10).

The permeability for the profile has been estimated at 10^{-4} ft/sec (Robert Pyke, personal communication). The uncorrected SPT, is about 10 blows/ft over most of the profile (Anderson and Tang, 1989).

6.B.3 Methods of Analyses

The purpose of the analyses is to compare the predictions of various approaches of one-dimensional site response analyses to each other, to recorded motions, and to synthetic motions at very high strain levels. As a result, an assessment of how well the various approaches model recorded motions, how similar or different they are at very high strain levels and an assessment of the adequacy of the vertical propagating shear-wave model would be produced.

The analysis methods chosen for study were DESRA (Lee and Finn, 1991), DYNA1D (Prevost, 1989), SUMDES (Li et al., 1992), TESS (Pyke, 1992), and RASCAL/SHAKE (Schnabel et al., 1972; McGuire et al., 1988). Time constraints precluded completion of the DYNA1D analyses; as a result it is not considered further in this report.

For the implementation of the nonlinear methodologies, the authors of the codes, if available, assisted in selecting appropriate parameter values based on the geotechnical models developed for each reference site. The information generally available was that contained in Tables 6.B-1 to 6.B-12 as well as average SPT data at the Treasure Island and Lotung LSST reference sites. In addition Dr. Robert Pyke provided estimates of friction angles for the liquefiable sands at Treasure Island and at Lotung. While the geotechnical models may not have been as complete as some code authors had desired, it was felt by the geotechnical engineers involved in the study that the nature, number, and types of tests performed represented an adequate basis for characterizing the dynamic response of the sites.

The process of conducting the comparative site response analysis study involved parameter selection by the code authors, initial analyses at all three reference sites, and then a review of the comparisons of predicted and observed motions by each author of their code's results. At this point, the authors were permitted to revise their input parameters. This interaction was to provide an important opportunity to

correct any significant errors in assigning parameter values and is a natural step in forward predictions where a number of analyses are done with different parameter values prior to adopting final best estimate predictions.

The engineers who participated in the code comparisons are R. Siddharthan for DESRA, J. Prevost for DYNA1D, X.S. Li for SUMDES, and R. Pyke for TESS.

The input files for all four computer codes for each of the sites are included following the figures at the end of this appendix.

6.B.3.1 DESRA-2C

Program DESRA-2C is an effective stress one-dimensional finite element site response analysis code which does not couple the wave propagation and diffusion equations. The code contains three main options: 1) total stress analysis ignoring the effects of seismically induced pore water pressures and strain hardening, 2) effective stress analysis with no redistribution of pore water pressure, and 3) effective stress analysis with redistribution and dissipation of pore water pressure. The DESRA-2C code implements a standard hyperbolic soil model using Masing's rules. Either a rigid or deformable base may be considered. The code was implemented as received.

6.B.3.2 RASCAL/SHAKE

Both the RASCAL and SHAKE codes represent an implementation of the equivalent-linear formulation (Seed and Idriss, 1969) applied to one-dimensional site response analyses. The RASCAL code is an RVT- (random vibration theory) based equivalent-linear approach which propagates the point-source outcrop power spectral density (Section 3.2) through a one-dimensional soil column. RVT is used to predict peak time domain values of shear strain based upon the shear-strain power spectrum. In this sense, the procedure is analogous to the program SHAKE (Schnabel et al., 1972) except that peak shear strains in SHAKE are measured in the time domain.

The purely frequency-domain approach of RASCAL obviates a time domain control motion and, perhaps just as significantly, eliminates the need for a suite of analyses based on different input motions. This arises because each time domain analysis may be viewed as one realization of a random process. In this case, several realizations of the random process are generally required to obtain a statistically stable estimate of site response. The realizations are usually performed by employing different control motions with approximately the same level of peak acceleration. In the frequency-domain approach, the estimates of peak shear strain as well as oscillator response are, as a result of the RVT, fundamentally probabilistic in nature.

In the current site response analyses, the theoretical outcrop power spectrum is replaced by the power spectrum computed from the recorded outcrop motions. The time histories computed by the RASCAL code are for comparisons to recorded motions and results of nonlinear analyses. They are computed by combining the phase spectrum of the control motion with the modulus and phase of the equivalent-linear transfer function. This process is done in a single step within the RASCAL code. Both random process estimates, from the computed power spectrum, and time domain solutions of the oscillator equation are produced for the response spectra. For the comparisons, only the RVT response spectra are used since these are the estimates used in computing the generic site amplification factors (Section 6).

6.B.3.3 SUMDES

The SUMDES code is a one-dimensional finite element vertical wave propagation code which can optionally treat horizontal and vertical motions simultaneously using a three dimensional hypoplasticity model. The solution solves the fully coupled wave propagation and diffusion equations. The formulation is effective stress (optional total stress) and the code can predict three directional motions as well as pore water pressure generation and dissipation. Either a rigid or compliant base may be used as well as a suite

(five) of constitutive models. An elasto-plastic model (one-dimensional) was used in the analyses. The code was implemented unmodified.

6.B.3.4 TESS

The computer program TESS is a one-dimensional explicit finite difference nonlinear site response analysis code which assumes vertically propagating shear waves and treats the wave propagation and diffusion equations as uncoupled. Either a rigid or compliant base may be specified. Since the effective stress formulation is used, the code can model excess pore pressure generation and dissipation. The soil model is hyperbolic (Hardin and Drnevich, 1972), and the Cundal-Pyke hypothesis (Pyke, 1979) is used for cyclic loading. The code was implemented unmodified.

6.B.4 Comparison of Analysis Methods using Recorded Motions

The following section shows the comparisons of recorded to computed motions at the surface of the reference sites for each computer code to the recorded motions. The comparisons are made to both time histories and 5% damped acceleration response spectra.

6.B.4.1 Gilroy 2

For reference site Gilroy 2 ground motions from the 1989 M 6.9 Loma Prieta earthquake are analyzed. Control motions are taken from the rock site Gilroy 1. Site distances and peak acceleration values are listed in Table 6.B-13 for the control motions.

The soil site-rock site station pair at Gilroy is especially significant in an assessment of the adequacy of the vertically propagating shear-wave model. The Gilroy strong motion accelerograph array begins with the rock site Gilroy 1 located just west of the western edge of the Santa Clara Valley. The remainder of the Gilroy array (2,3,4,6, and 7) extends roughly eastward across the valley at 2–3 km intervals with Gilroy 7 founded on shallow soil on the eastern edge of the valley. The soil depth varies from zero at Gilroy 1 to about 600 ft at Gilroy 2 (2 km east) and to several thousand feet at Gilroy 3, 2–3 km east of Gilroy 2. The site at Gilroy 2 then is located on the edge of a steeply dipping bedrock interface, the area or zone where two-dimensional basin effects are predicted to be most pronounced (Silva, 1991).

6.B.4.1.1 Linear Analyses. The linear analyses were conducted using the program RASCAL with low strain modulus and damping values. In the linear analyses both small and large motions are considered. The small motions are from recordings of aftershocks made at Gilroy 1 and Gilroy 2 (Silva and Stark, 1991). Magnitudes were in the range of 2–4 at distances of 10–30 km with resulting low levels of ground motions. Figure 6.B-9 shows the average transfer function (Gilroy 2/Gilroy 1) based on Fourier spectra computed from recordings of 13 aftershocks (solid line).

Also shown is the theoretical transfer function (dashed line) computed assuming vertically incident shear waves using the base case shear-wave velocity profile (Figure 6.B-1) and the low-strain damping (3%) from the damping curves shown in Figure 6.B-3. The transfer function computed from the recordings is truncated at 1 Hz, the seismometer corner frequency, so the fundamental near 1 Hz is not present. At high frequencies, the truncation is at 20 Hz just before noise begins to flatten the transfer functions. A resonance shown in the recorded motions near 2 Hz indicates that the velocities in the profile are too high since the corresponding resonance in the computed motions is near 2.2 Hz. The next four resonance peaks, near 3 Hz, 5 Hz, 6 Hz and 10 Hz show a similar shift. This trend suggests that the velocities in the linear model should be lowered about 20%. Revising the profile primarily in the upper 300 ft shifts the peaks to coincide more closely with those in the empirical transfer function.

Figure 6.B-10 shows the transfer function computed with the revised profile (Figure 6.B-11). As the figure indicates, the match is significantly improved but general overpredictions in the frequency range of about 3–8 Hz and near 1 Hz remain. These departures may be due to two or three-dimensional effects. However the overall fit is considered quite good, particularly since a range in source azimuth and depth is sampled

in the 13 aftershocks. Also the match in falloff at high frequencies suggests that the overall small strain damping for the profile of 3% ($Q_s = 17$) is about correct.

The revised shear-wave velocity profile shown in Figure 6.B-11 reflects the lower velocities as well as a gradient rather than a sharp boundary at a depth of about 150 ft. While this revised profile is compatible with the in situ geophysical data (Figure 6.B-2) it does favor the lower velocity observations particularly at depths near 50 ft and 150 ft. Since the lower velocity observations tend to be associated with the down-hole measurements (Appendix 8.A) which are at lower frequencies (30–40 Hz), the difference may be attributable to dispersion effects.

Interestingly and most importantly, equivalent-linear analyses with both the base-case and revised profiles for the Loma Prieta earthquake (nearly 50%g control motions) resulted in very little difference in computed response (10% maximum) in either time histories or response spectral ordinates. While the shear-wave profile has a large effect on small motions, for strong ground motions the effect of the initial profile is significantly less. This suggests that for site response analyses for strong motion, details of the profile may be less important than proper characterization of the strain dependencies of the material properties. That is, for small earthquakes, the linear or initial velocity and damping profile controls the response while for larger motions, the strain compatible properties are more important. More significantly, iterated properties are not highly sensitive to the small-strain or linear profile. This result is consistent with nonlinear site response (soil and rock) being a significant factor in the trend of a reduction in uncertainty with increasing magnitude shown in strong motion data (Aki, 1988). In view of the relative insensitivity of the strong motion analysis results to details in the shear-wave velocity profile, the base case profile is used in all analyses.

Results of the linear analysis of motions from the Loma Prieta earthquake using small strain material properties are shown in Figure 6.B-12 for both components of motion at Gilroy 1 and Gilroy 2. As expected the motions are dramatically overpredicted by over a factor of two at high frequencies. The frequencies of the resonances are matched reasonably well by the linear analysis suggesting that the effects of increasing damping with increasing strain are more significant than the accompanying reduction in velocity if the equivalent-linear and nonlinear analyses are to result in an improved comparison.

6.B.4.1.2 Equivalent-Linear Analysis: RASCAL. The equivalent-linear analysis results are shown in Figure 6.B-13. The reduction in the high frequency energy content is obvious in the response spectra, resulting in a greatly improved fit. Over the entire frequency range the fit is generally quite good, with the predicted PGA values within 30% of the observed on one component and within 1% on the other. The average shear-wave strain iterated damping for the entire profile is about 7%, more than double the small strain value of 3%.

The effects of the dipping structure do not seem to be significant even at very low frequencies. The generation of small amplitude surface waves by the dipping interface may be manifested in the increase in coda shown in the recorded motions relative to the motions computed by vertically propagating the Gilroy 1 rock outcrop control motions. If this is indeed the case, it illustrates an important point in that care should be exercised in demonstrating the effects of dipping interfaces on strong ground motions using linear analyses. The higher damping during the strong shaking may be severely damping these secondary scattered wavefields rendering their amplitudes much smaller than a linear two-dimensional analysis with small strain damping would suggest. Naturally this also applies to observations of two- and three-dimensional effects using low levels of recorded motions.

6.B.4.1.3 Nonlinear Analyses. In the following sections, results of the nonlinear analyses using the codes DESRA-2C, SUMDES, and TESS will be presented.

6.B.4.1.3.1 DESRA-2C. Results from the DESRA-2C analysis are shown in Figure 6.B-14. The north component shows a general underprediction over most of the frequency range while the east component shows a much more favorable match. In general however, the tendency is toward overall

underprediction. The time histories appear much more similar to the equivalent-linear RASCAL analysis than do the response spectra. This indicates that the more robust measure of goodness-of-fit is likely to be made on spectral ordinates if timing of arrivals is of secondary importance to correct amplitudes.

6.B.4.1.3.2 SUMDES. Figure 6.B-15 shows the results from the code SUMDES. Although the north component shows some underprediction, the comparison to the empirical spectra is quite good. The computed time histories show comparable frequency content and durations to recorded motions as well as similar coda wave levels. The increase in coda levels over the RASCAL and DESRA-2C analyses indicate that the effects of the dipping structure are actually quite minimal since the predicted coda amplitudes now match the observed. Perhaps overdamping in the RASCAL and DESRA-2C analyses was the cause of reduced coda relative to the recorded motions.

6.B.4.1.3.3 TESS. The results from the TESS analysis are shown in Figure 6.B-16 and are very similar to the SUMDES analysis. There is a slight underprediction of the north component with a very good match to the east component. The overall frequency content, durations, and coda levels compare very favorably with the recorded motions.

6.B.4.1.4 Summary of Gilroy 2 Analyses. As a final comparison, log average response spectra of the horizontal components for both recorded and computed motions are shown in Figure 6.B-17. DESRA shows a general underprediction over much of the frequency range while RASCAL provides a good overall match with an overprediction of the resonance near 3 Hz. This is not unexpected in equivalent-linear analyses since the velocities are constant at the strain compatible values, while in nonlinear analyses they vary with time. As a result, resonance phenomena are less well pronounced in nonlinear analyses at high strain levels and may, in fact, depend upon the character (duration) of the control motion.

In an overall sense the comparisons are considered quite favorable with RASCAL, SUMDES, and TESS showing generally better agreement to recorded motions. The vertically propagating shear-wave model appears to result in very favorable comparisons to recorded motions. Both equivalent-linear and nonlinear analyses show very similar results and an increase in damping, from 3% to about 7%, with increasing levels of motion required to model both weak and strong motions.

6.B.4.2 Treasure Island

As with reference site Gilroy 2, ground motions from the 1989 M 6.9 Loma Prieta earthquake are analyzed. Recordings at Yerba Buena Island are used as adjacent (≈ 2 km) rock outcrop control motions. Table 6.B-13 lists the control motion peak accelerations and source-to-site distance.

6.B.4.2.1 Linear Analyses. Figure 6.B-18 shows the small-strain transfer functions (Treasure Island/Yerba Buena Island) computed from recordings of 7 aftershocks (Jarpe et al., 1989). Also shown is the computed transfer function using the base case shear-wave velocity profile (Figure 6.B-4). The empirical transfer function exceeds the computed by a factor of nearly three for frequencies up to about 7 Hz. The locations of the resonances are predicted quite well by one-dimensional linear analysis out to the same frequency. Because the shapes in the empirical and theoretical transfer functions show good agreement out to about 7 Hz, it is difficult for the effects of basement topography to be the cause of the largely constant offset over this frequency range.

The sharp drop in the empirical ratio near 7 Hz is due largely to a sharp drop in the Treasure Island Fourier amplitude spectra. This may be a result of unmodeled shallow soil structure since the boreholes were not placed at the exact location of the recording instrument. The only remaining sources of the discrepancy are the control motions, perhaps being deficient for frequencies up to about 7 Hz or high in energy beyond 7 Hz relative to the basement of Treasure Island, and the impedance contrast from soil to rock at Treasure Island. The control motions could simply be inappropriate due to topographic effects at Yerba Buena Island although this is not obvious from visual examination of the Fourier amplitude spectra. The basement shear-wave velocity was taken as 3500 ft/sec (Figure 6.B-4) based upon suspension

logging results (Figure 6.B-5) (Appendix 8.A). If a large velocity gradient exists below the existing hole such that shear-wave velocities of about 10,000 ft/sec were reached within 100–200 ft, results of analyses indicate that the computed ratio would move up to the level of the empirical. It should also be pointed out that this process assumes that the same gradient does not exist under the recording site at Yerba Buena Island.

This large relatively constant difference of a factor near three in the empirical and theoretical transfer functions is an important issue. A number of researchers (Idriss, 1990; Dickenson et al., 1991; Hryciw et al., 1991) have used this soil/rock station pair with similar profiles in site response studies. The results have been used to adjust model parameters and make inferences regarding the ability of different procedures to model recorded motions as well as predict liquefaction. Until this issue is resolved by recording small earthquakes at the surface, at some depth in the basement at the Treasure Island site, and at the Yerba Buena site, as well as extending the deep hole further into basement rock and measuring velocities, uncertainty will remain in interpreting results in site response analyses using the strong motion recordings from these sites.

The linear analysis results for the Loma Prieta earthquake are shown in Figure 6.B-19. The computed spectra show a general underprediction up to about 2–3 Hz and match the recorded motions reasonably well beyond. Of interest, the predicted spectral peaks occur at higher frequencies than in the recorded strong ground motions but are matched very well in the weak motion transfer functions.

The computed time histories reflect the low frequency deficiency shown in the spectra, particularly the east component, and match the observed peak accelerations quite well. It is important to point out, in examining the recorded and computed time histories, that no adjustment has been made for absolute time. For example, Figure 6.B-19 suggests that the predominant motion in the computed motions arrives before that of the recordings. Neither the difference in trigger times of the recorders at Treasure Island and at Yerba Buena Island nor the differential travel times has been accounted for.

The broad band nature of the underprediction, seen now down to 0.1 Hz, likely reflects the same phenomenon as exhibited by the small-strain Fourier spectral ratios and further suggests a large scale feature such as the impedance contrast or a steep velocity gradient at the base of the Treasure Island profile.

6.B.4.2.2 Equivalent-Linear Analysis: RASCAL. Figure 6.B-20 shows the results of the equivalent-linear analysis. As expected very little difference from the linear analysis is seen in the computed spectra below about 1 Hz. The fundamental resonance is at about 0.7 Hz and is clearly seen on both components. The higher resonances are shifted to lower frequencies in the equivalent-linear analysis and more closely match the empirical spectra, particularly on the larger of the two components (east). The underprediction now extends over most of the frequency range due to the increased damping in the profile associated with the equivalent-linear analysis. This is reflected in the time histories as well, showing less high frequency energy and reduced peaks. Increasing the control motion uniformly would result in a direct (one-to-one) enhancement of low frequencies but, due to resulting increase in strain levels and accompanying higher damping, show less of an effect at high frequencies. Increasing the impedance contrast at the soil rock interface would have such an effect.

Other workers have obtained different results at this site using similar methodologies with different material properties, modified control motions, and by varying sensitive parameters in the computer codes. This points out that better or worse comparisons may be obtained for earthquakes which have occurred. The intent of the present analyses was to perform, to the extent possible, a forward prediction at each site using measured material properties and implementing the analysis procedures in a consistent manner. The results then provide a more rational basis for evaluating how well the prediction methodologies perform.

6.B.4.2.3 Nonlinear Analyses

6.B.4.2.3.1 DESRA-2C. Figure 6.B-21 shows the results from the DESRA-2C analysis. The computed response spectra show an overall underprediction to the observations, similar to the RASCAL results. The motions computed with DESRA-2C show more low frequency energy on the east component and less high frequency energy on the north component. These trends are reflected in the computed time histories as well. In general however, the DESRA-2C and equivalent-linear results are comparable.

6.B.4.2.3.2 SUMDES. Results from the SUMDES analysis are shown in Figure 6.B-22. For the north component, the SUMDES motions show slightly higher levels while the east component motions are significantly higher. The spectrum computed for the east component matches very well the observed up to about 1.3 Hz and beyond 10 Hz, missing the resonances at 3 and at 7 Hz. The time history for the east component matches the observed quite well with only a 7% underprediction in peak acceleration. As with the previous analyses, however, the amplitude, duration, and character predicted for the north component does not match the observed very well.

6.B.4.2.3.3 TESS. The results for the TESS analysis are shown in Figure 6.B-23 and indicate similar features to the other analyses. A general broad-band underprediction of motions is shown for both components and a poor characterization of the waveform for the north component are seen.

6.B.4.2.4 Summary of Treasure Island Analyses. A comparison of the average spectra is shown in Figure 6.B-24. In the figures, the overall broad-band nature of the underprediction is quite apparent. All four analyses produce comparable levels of computed motions when the components are averaged. These results are in accord with the Gilroy 2 analyses and suggest that the equivalent-linear and nonlinear analyses produce very similar strong ground motion predictions for these levels of motion and soil conditions. For the Treasure Island site, results from the equivalent-linear and nonlinear analyses are comparable to the linear analysis, indicating that nonlinear soil effects, in terms of surface ground motions, were not significant during the Loma Prieta earthquake. This is in accord with the observations of Silva and Stark (1992).

The similar pattern in broad band underprediction suggests that the Yerba Buena control motion may be inappropriate as input to the Treasure Island profile, perhaps due to topographic effects. Alternatively, or in conjunction, a strong velocity gradient may exist in the bedrock beneath the Treasure Island site that is not present at the Yerba Buena site. Until this issue is resolved, results of analyses with these recordings will have uncertainties regarding inferences on site response analysis procedures and nonlinear soil models.

6.B.4.3 Lotung

At the Lotung reference site data from a vertical array of three component accelerometers are used. Sensor packages are located at the surface and depths of 20 ft (6 m), 36 ft (11m), 56 ft (17 m), and at 154 ft (47 m) (Anderson and Tang, 1989). Data from three earthquakes are analyzed, two of which represent strong motion with surface peak acceleration values near 10%g (Table 6.B-13, LSST events 7 and 16). A third earthquake (LSST event 10), with an average surface peak acceleration value of about 4%g is used for a small-strain analysis to assess the appropriateness of the base-case or low strain profile (velocity and damping) and the vertically propagating shear-wave assumption. Events 7 and 10 have recordings at all 5 levels while for event 16 recordings are available from the top 4 levels (surface, 20 ft, 36 ft, and 56 ft). Interestingly, even at these low levels of motion, the profile appears to have gone nonlinear, which is consistent with the results of Silva et al. (1990).

For the strong motion analyses, LSST events 7 and 16 are used as control motions. Analyses consist of propagating the motions recorded at the deepest sensor package to the shallower levels. For LSST event 7 the recordings at 154 ft are used while for LSST event 16, the deepest recordings are at the 56 ft level. Results are presented for analyses from the deepest recording levels to the surface and then to progressively deeper levels. For example, LSST event 7 has motions computed at the surface, 20 ft, 36 ft, and 56 ft, all using control motions recorded at 154 ft.

6.B.4.3.1 Linear Analysis. The small-strain transfer functions computed for LSST event 10 using a linear analysis are shown in Figures 6.B-25 to 6.B-28 for surface/154 ft, 20 ft/154 ft, 36 ft/154 ft, and 56 ft/154 ft. The control motion is taken from the recordings at 154 ft for all cases (Table 6.B-13). Although the peaks are matched reasonably well for all the ratios out to about 10 Hz, indicating an appropriate shear-wave velocity profile and wave propagation model, the predicted amplitudes are high, particularly for the higher-order resonances. The small strain damping was fixed at 1% for the profile based on the damping curves (Table 6.B-12, $10^{-4}\%$ shear strain) and the computed ratios indicate that this value may be too low.

The small-strain analysis was repeated using the equivalent-linear approach and the resulting transfer functions are shown in Figures 6.B-29 to 6.B-32 for the four depth ratios. The equivalent-linear transfer functions show a significant reduction in amplitude and a slight shift in predominant frequencies. The overall fits, particularly the surface/154 ft ratios, are measurably improved. The average damping throughout the profile increased from 1.0 to 1.7% which suggests that at soft sites, nonlinear soil response can have effects even at surface acceleration levels of 4%g.

To explore the effects of increased levels of recorded motions, an analysis of the frequencies and amplitudes of the fundamental resonance peaks was done for the three earthquakes studied (LSST events 7, 10, and 16). The surface-to-depth ratios (20, 36, 56 and 154 ft) were analyzed and the results are shown in Table 6.B-14. The trend in reduced frequencies and amplitudes of the fundamental resonance with increasing levels of motion (0.035g, 0.080g, and 0.090g) is clear. There is more than a factor of two shift in frequency and amplitude at all levels except surface/154 ft. For this ratio, the deeper soils have higher velocities, develop lower strain levels, and the frequency shift and amplitude reduction are correspondingly less. These results present a compelling case for in situ nonlinear soil response and suggest that these effects could extend to large source-to-site distances from large to moderate earthquakes for soft sites.

Results from the linear analyses using the small strain material properties are shown in Figures 6.B-33 to 6.B-36 for LSST event 7 and Figures 6.B-37 to 6.B-39 for LSST event 16. From the figures, the linear catastrophe is obvious, and the small strain damping of 1% based upon the laboratory testing results (Appendix 8.B) is far too small. The computed motions show a large fundamental resonance, approaching 8g in spectral acceleration in some cases, and the time histories ring at the resonant frequencies (complete with wrap around effects). Setting the damping to the level of LSST event 10, 1.7%, would not significantly alter the results. Clearly these soils are behaving in a strongly nonlinear way and represent a severe test of the analysis procedures as well as the laboratory-derived material strain dependencies.

6.B.4.3.2 Equivalent-Linear Analysis: RASCAL. Results of the equivalent-linear analyses for LSST events 7 and 16 are shown in Figure 6.B-40 to 6.B-43 and Figures 6.B-44 to 6.B-46. For both earthquakes and at all levels, the predicted spectra agree in overall shape and level with the empirical. The resonances tend to be overpredicted in most cases, particularly for event 7, but the agreement between the predicted and empirical spectrums is generally quite good. The time histories generally reflect the correct amplitudes and durations and match the recorded peak acceleration levels quite well. The vertically propagating shear-wave model and laboratory derived material strain dependencies appear to provide an accurate representation of the response of this profile throughout the upper 154 ft.

6.B.4.3.3 Nonlinear Analyses

6.B.4.3.3.1 DESRA-2C. The DESRA-2C results for both LSST events 7 and 16 are shown in Figures 6.B-47 to 6.B-50 and Figures 6.B-51 to 6.B-53. For event 7, the motions computed at the surface and at 20 ft are very close to the empirical, except for the low response for frequencies above 3–4 Hz for the north component at the surface. For the greater depths (36 and 56 ft, Figures 6.B-49 and 6.B-50) there is a tendency for overprediction above about 1 Hz.

For event 16, there is a general underprediction at the surface above about 0.4 Hz and a slight underprediction at a depth of 20 ft. The match between recorded and computed motions is much closer at a depth of 36 ft, the shortest propagation distance.

In general, the results of DESRA-2C are not as good as those using the equivalent-linear analysis technique. This is opposite to the results of Chang et al. (1990), although the differences between the equivalent-linear and nonlinear results were much less than is shown here. Chang et al. used a modified version of the DESRA code by incorporating a Martin-Davidenkov soil model to analyze event 7. The study also used a different soil profile as well as different modulus reduction and damping curves. Additionally in the Chang et al. study, the control motion was at 56 ft, rather than the 154 ft level used here. As a result, the propagation paths are much shorter. The use of deeper motions reduces the possible contamination of the control motion by surface generated scattered wavefields (Silva et al., 1988). As a result of these differences, it is difficult to compare the results of the two studies in a meaningful way, and it would be of interest to repeat the present study using the Martin-Davidenkov soil model.

6.B.4.3.3.2 SUMDES. The corresponding results of the SUMDES nonlinear analyses are shown in Figures 6.B-54 to 6.B-57 and Figures 6.B-58 to 6.B-60. For LSST event 7, the results are similar to DESRA-2C at the surface, showing an excellent match to the east component and an underprediction on the north component above around 3 Hz. For the at-depth motions, however, the SUMDES results show a general underprediction for frequencies exceeding about 2–3 Hz. The trend is reflected in the time histories where the predicted peak accelerations are less than the recorded motions even though the general character of the motions match very well.

The results for event 16, on the other hand (Figures 6.B-58 to 6.B-60), show excellent agreement with the recorded motions at all levels. The match in the spectral ordinates is quite good and the time histories, for most components, show an extraordinary match.

6.B.4.3.3.3 TESS. For LSST event 7, the results for TESS are shown in Figures 6.B-61 to 6.B-64. With the exception of the surface north component above about 7 Hz, the match in response spectra and time histories is quite good and comparable to the SUMDES results for event 16. The overall shapes of the response spectra agree very well with the empirical and even match in detail in several cases. The computed time histories show similar close agreement matching the amplitudes, duration, and general character of the recorded motions.

The corresponding results for event 16 are shown in Figures 6.B-65 to 6.B-67. For this earthquake, the match is also quite good but degraded somewhat compared to the results of event 7. A tendency exists for a broad-band overprediction of the east component at all three levels (surface, 20 ft, and 56 ft). The computed motions for the north component match the observed motions extraordinarily well, showing only minor departures from the empirical response spectrums. The time histories show an equally good match as well.

6.B.4.3.4 Summary of Lotung Analyses. A significant aspect of the Lotung analyses is a suggestion of nonlinear soil response at very low levels of surface peak acceleration (4%g), as well as a clear demonstration of consistent frequency shifts and decreases in resonance peaks with increasing levels of motion. These results, when considered with the linear analyses of events 7 and 16 show that a large increase in shear-wave damping is taking place in going from surface accelerations of 4%g to levels of 8% to 9%g. Results of the equivalent-linear and nonlinear analyses for the same earthquakes, summarized in the average response spectra shown in Figures 6.B-68 to 6.B-71 and Figures 6.B-72 to 6.B-74, indicate that the laboratory derived dynamic material properties reflect the in situ strain dependencies very well. The average spectra also show, for both earthquakes, very similar results for the equivalent-linear and nonlinear analyses. The individual component analyses showed that some approaches performed better than others for certain components, depths, and earthquakes but, on average, each approach produces comparably good results.

The vertically propagating shear-wave model represents an accurate representation of the predominant motion at this site over the frequency range of 0.1 to over 30 Hz for the three earthquakes studies. In addition, the equivalent-linear results are generally comparable to those of the nonlinear analyses with a tendency to overpredict the resonances, particularly the fundamental. All of these results are consistent with those of the previous sites: Gilroy 2 and Treasure Island.

6.B.5 Comparison of Analysis Methods using Artificial Motions

The comparisons of analysis methods using recorded motions provided valuable information on methods as well as a validation of the wave propagation model and laboratory testing results. Since these analysis procedures are implemented in forward predictions for sites located very close to potential large earthquake sources, it is also of interest to further compare results at very high levels of control motion.

6.B.5.1 Characterization of Control Motions

To provide a comparison over a wide range in motions, synthetic rock outcrop control motions were generated for peak acceleration levels of 50%, 75%, 100%, and 125%g. The stochastic ground motion model (Section 3) was used with standard Western North America parameters (Table 6.B-15) (Boore, 1986; Silva and Darragh, 1993) and a Wald et al. (1991) crustal model (Schneider et al., 1993) to generate the Fourier amplitude spectrum for the control motions. Table 6.B-15 shows the magnitudes, distances, peak values, and predominant frequencies for the synthetic control motions. The peak values and predominant frequencies are random process estimates (Boore, 1983). To generate a time history for the nonlinear analyses, the phase spectrum from a recording at the Southern California Edison Lucerne site of the M 7.5, 1992 Landers earthquake was combined with the appropriate theoretical amplitude spectrum (Silva and Lee, 1987).

Use of the theoretical stochastic Fourier amplitude spectrum to generate different levels of motion, rather than scaling a single time history, retains more closely the proper distribution of low- and high-frequency energy content (scaling) expected in empirical recordings. Maintaining the same phase spectrum permits comparisons using different control motion levels but retains the same energy distribution in time.

6.B.5.2 Characterization of Site Model

Category 3 from the generic profiles used in computing the soil amplification factors for Eastern North American sites is taken as the reference profile (Section 6). The shear-wave velocity profile for this category is shown in Figure 6.B-75. The velocity is about 1000 ft/sec at the surface and increases rapidly to 2000 ft/sec at the bottom of the profile (120 ft). The bedrock velocity is 6000 ft/sec which is consistent with the median bedrock velocity in Eastern North America (Section 6). The soil is assumed to consist of clean sands with the modulus reduction and damping curves shown in Figure 6.B-76. For the nonlinear methods, total stress analyses were used since the interest is in surface ground motions and the profile is quite stiff.

6.B.5.3 Results of Analyses

The equivalent-linear from the RASCAL analyses results for control motions of 0.50 and 0.75g are shown in Figure 6.B-77 and for 1.00 and 1.25g in Figure 6.B-78. At all levels of control motion, the equivalent-linear results show amplification for frequencies above about 1 Hz and particularly at the resonant frequencies. The fundamental soil resonance is close to 3 Hz which is also near a resonance in the Wald et al. (1991) crust. As the control motion increases from 0.50 to 0.75g, the soil resonance shifts to a slightly lower frequency and closer to the crustal resonance resulting in greater amplification. The time histories reflect the soil resonance as well showing increased monochromatic energy for motions with high resonant amplification. Interestingly, the control motions also reflect the crustal resonance showing a small burst of 3 Hz energy arriving near 16 sec. The amount of amplification decreases with increasing control motion levels due to the increased damping but the effect is not dramatic. The overall stiffness of the profile is such that the equivalent strains do not build up to very high levels.

Results from the DESRA analyses are shown in Figures 6.B-79 to 6.B-80. The response spectra indicate similar levels of motion to the RASCAL results with slightly more amplification at low frequencies and less pronounced resonances. The time histories reflect the similarities to the equivalent-linear results with slightly less ringing but similar overall levels of motions.

In the SUMDES analyses, Figures 6.B-81 to 6.B-82, the computed soil motions appear to be slightly amplified over most of the frequency range. As with DESRA, the peak accelerations are amplified at 0.50g and deamplified at higher levels of control motion.

The computed time histories, for the two higher motions of 1.00 and 1.25g clearly show a nonlinear saturation effect with flattened tops. The sharp corners on the waveforms imply high frequency energy content and may be a consequence of the nonlinear soil model at high strain levels.

The corresponding results for the TESS analyses are presented in Figures 6.B-83 to 6.B-84. In general TESS shows results similar to those of SUMDES with slightly more amplification. The time histories however, do not display the same blocky character for the highest control motions, 1.00 and 1.25g, possibly reflecting different assumptions in the nonlinear soil model.

In order to view the results of the different analysis techniques adjacent to one another, Figures 6.B-85 to 6.B-88 show the computed response spectra plotted on one figure for each level of control motion. The figures show the general similarity of the computed motions with the equivalent-linear motions showing more pronounced resonances and slightly higher motions in most cases.

These results suggest that the equivalent-linear analyses, for the generally stiff soil profiles used to compute the amplification factors, produce ground motions that are in good agreement with those resulting from fully nonlinear analyses. Computed amplification factors using the equivalent-linear methodology implemented in the RASCAL code can then be expected to be in general accord with fully nonlinear results and may be slightly conservative for frequencies of 1 Hz and above. Predicted motions at resonance peaks will tend to be particularly conservative.

6.B.6 Conclusions

The major issues in predicting the effects of site response to strong ground motions include a suitable wave propagation model, the in situ strain dependencies of dynamic material properties, and how the effects of material nonlinearities are treated computationally. In the work presented here, the intent was to treat all three aspects by comparing observed strong ground motions to predicted motions at three carefully characterized reference sites. Reasonably comprehensive geotechnical models including laboratory testing were developed for Gilroy 2, Treasure Island, and the Lotung, Taiwan LSST site. The sites possess all of the features which were thought necessary to provide a good validation: recordings of both high- and low-strain ground motions, a dipping interface at least at one site (Gilroy 2), a wide range in material properties from sands and gravels to soft silts and stiff clays, deep (70 ft) and shallow (surface) water tables, and a wide range in velocities from deep and stiff at Gilroy 2 to shallow and very soft at Lotung.

The analyses procedures compared all used the vertically propagating shear-wave model and included the equivalent-linear, implemented through the RVT based RASCAL code, and three nonlinear methodologies DESRA, SUMDES, and TESS. Results using small-strain data and linear analyses show low-strain laboratory damping measurements to be consistent with transfer functions calculated using recordings from small earthquakes. For strong ground motions, and even weak motions at the Lotung LSST site, nonlinear soil response was observed and modeled very well by both the equivalent-linear and nonlinear techniques. Both computational approaches to model the effects of soil nonlinearity produced equally good comparison to recorded ground motions both in response spectra and time histories.

Typical run times are shown in Table 6.B-16 and range from about 60 sec for TESS to 700 sec for SUMDES with DESRA-2C and RASCAL at around 100 sec. The SUMDES runs require significantly longer times because it is solving the fully coupled set of wave propagation and diffusion equations. The RASCAL run time also reflects the RVT computation of about 90 response spectral ordinates, not included in the nonlinear analyses. The general conclusion is that nonlinear and equivalent-linear codes require about the same execution time unless a fully coupled nonlinear analysis is performed.

In both the small- and large-strain analyses at the soil/rock site pair (Gilroy 2/Gilroy 1) with a known dipping interface between, 2- or 3-dimensional effects were shown to be small and not considered in modeling the site response from either the Loma Prieta earthquake or recordings of aftershocks. Results from the simple vertically propagating shear-wave model provided a good comparison to observed motions from 0.1 Hz to over 30 Hz for the mainshock and from about 2 Hz to 20 Hz for the aftershocks (the bandwidth of useable data).

At reference site Treasure Island, a large discrepancy exists between the computed transfer function using the base case shear-wave velocity profile and the empirical transfer function from aftershocks recorded at Treasure Island and Yerba Buena Island. The empirical transfer function exceeds the analytical by a factor of about 2–3 for frequencies up to nearly 10 Hz. Nonlinear soil response does not appear to be the cause of this discrepancy as the computed motions for the Loma Prieta earthquake using the Yerba Buena recordings as control motions show a broad-band and general underprediction of the recorded motions. It was suggested that either or both a topographic effect at Yerba Buena Island or a large velocity gradient beneath the level of measured velocities at Treasure Island may be responsible. Until this issue is resolved, uncertainties will remain regarding analyses done with this pair of strong motion recordings.

Additional analyses were performed using synthetic control motions to examine the sensitivities of the codes to very high levels of motions. A hypothetical 120 ft soil column of clean sands was analyzed for control motions of 0.50, 0.75, 1.00, and 1.25g. In general, the equivalent-linear and nonlinear analyses showed very similar results. The equivalent-linear analyses resulted in uniformly higher motions than the nonlinear analyses in most cases, particularly at the column resonant frequencies, indicating that, for stiff profiles, equivalent-linear site response analyses tend to be slightly conservative.

The general conclusion resulting from these analyses is that conventional one-dimensional site response analyses incorporating nonlinear soil behavior based upon careful laboratory testing and with reasonably accurate soil profiles can accurately predict the effects of soils on strong ground motions.

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Table 6.B-1

Base-Case Profile for Gilroy 2

Thickness (ft)	Vs (ft/s)	Vp (ft/s)	Total Unit Weight (pcf)
35.00	650.00	1000.00	120
10.00	1000.00	1600.00	120
30.00	1560.00	2800.00	120
31.00	1000.00	5360.00	120
20.00	1140.00	5360.00	133
12.00	1230.00	5640.00	133
112.01	2100.00	6750.00	133
19.98	2100.00	6000.00	133
23.00	1550.00	5200.00	133
29.00	1730.00	6000.00	133
238.01	2300.00	6835.31	133
00.00	3900.00	10500.00	144

Table 6.B-2

Geologic Log For Gilroy 2 (water table at 70 ft)

Depth to Top (ft)	Description
0	fine to very fine SAND and sandy LOAM with some gravel lenses
19	silty clay LOAM
30	loamy, fine to medium SAND
39	well-sorted, coarse SAND with some gravel
43	very stiff CLAY
48	SAND
50	moderately well-sorted, sandy fine GRAVEL and pebbles
70	very stiff CLAY
103	fine loamy SAND
112	silty CLAY
126	loose loamy SAND
130	dense, moderately well-sorted, fine gravelly SAND
150	GRAVEL
155	gravelly SAND
166	GRAVEL
185	gravelly SAND
194	sandy CLAY
201	GRAVEL
205	very poorly sorted gravelly SAND
215	GRAVEL
228	gravelly SAND with sandy clay lenses
246	gravelly clayey SAND, very poorly sorted with gravel lenses
304	clayey SAND
310	gravelly SAND
314	clayey SAND and sandy CLAY
354	gravelly, clayey SAND, very poorly sorted, very dense, with thin gravel beds
384	clayey SAND
388	clayey, gravelly SAND and clay LOAM, very poorly sorted
471	GRAVEL
476	sandy, clay LOAM with gravel
536	GRAVEL
540	sandy clay LOAM with gravel
550	deeply weathered SILTSTONE (Monterey Shale)
577	sandy CLAY (serpentinite)
790	bottom of hole

Table 6.B-3
Properties of Samples from Gilroy 2

Sample No.	Sample Depth ft (m)	Soil Description	Liquid Limit %	Plasticity Index %	Water Content %	Total Unit Weight pcf (g/cc)	Void Ratio e	Dry Unit Weight pcf (g/cc)	Specific Gravity Gs	Degree of Saturation %
G1	10 (3.0)	Dark Brown Clayey Silt with Sandy Material	29	7	26.1	117.1 (1.88)	0.78	93 (1.49)	2.70	86
G2	20 (6.1)	Dark Gray Silty Clay	43	23	30.0	118.8 (1.90)	0.84	91,96 (1.46,1.54)	2.70	96
G3	50 (15.2)	Silty Sand with Gravel	-----	-----	15.8	123.4 (1.98)	0.58	107 (1.71)	2.65	76
G4	85 (25.9)	Light Gray Stiff Clay with Horiz. Bedding	47	17	30.8	121.0 (1.94)	0.82	93,95 (1.49,1.52)	2.70	100
G5	120 (36.6)	Silty Sand	-----	-----	19.8	134.1 (2.15)	0.55	112,110 (1.79,1.76)	2.65	100
G6	210 (64.0)	Gravelly Sand	-----	-----	14.8	130.1 (2.08)	0.52	124 (1.99)	2.65	62
G7	420 (128.0)	Gravelly Clay	-----	-----	28.0	136.4 (2.19)	0.58	113 (1.81)	2.65	85
G8	348 (106.1)	Clayey Silt	35	13	23.7	127.7 (2.05)	0.60	103 (1.65)	2.70	100
G9	420 (128.0)	Gravelly Clay	-----	-----	14.0	136 (2.18)	0.41	120 (1.92)	2.70	92

Table 6.B-4

Modulus-Reduction and Damping Curves for Gilroy 2

Strain (%)	G/GMAX (0-40 ft)	G/GMAX (40-80 ft)	G/GMAX (80-130 ft)	G/GMAX (130 + ft)
1.0E-4.0	1.00	1.00	1.00	1.00
1.0E-3.5	1.00	1.00	1.00	1.00
1.0E-3.0	1.00	1.00	1.00	1.00
1.0E-2.5	0.99	0.99	1.00	0.95
1.0E-2.0	0.91	0.93	0.95	0.81
1.0E-1.5	0.72	0.76	0.80	0.57
1.0E-1.0	0.47	0.51	0.56	0.32
1.0E-0.5	0.24	0.27	0.31	0.14
1.0E-0.0	0.09	0.11	0.14	0.05

Strain (%)	Damping % (0-40 ft)	Damping % (40-80 ft)	Damping % (80-130 ft)	Damping % (130 + ft)
1.0E-4.0	3.0	3.0	3.0	3.0
1.0E-3.5	3.0	3.0	3.0	3.0
1.0E-3.0	3.0	3.0	3.0	3.3
1.0E-2.5	3.5	3.3	3.2	4.1
1.0E-2.0	4.7	4.4	4.1	6.1
1.0E-1.5	7.5	6.9	6.3	10.0
1.0E-1.0	12.1	11.2	10.1	15.9
1.0E-0.5	18.3	17.3	16.0	22.2
1.0E-0.0	24.4	23.5	22.3	27.5

Table 6.B-5

Base-Case Profile for Treasure Island

Thickness (ft)	Vs (ft/s)	Vp (ft/s)	Total Unit Weight (pcf)
8.20	575.00	1425.00	120
18.05	438.00	5390.00	120
16.77	583.53	5390.00	120
17.00	583.53	4055.32	110
9.97	680.00	4055.32	110
24.51	540.00	4055.32	110
15.52	1041.00	5347.00	114
40.00	877.00	5347.00	114
56.00	877.00	5347.00	128
40.00	877.00	5347.00	115
42.69	1266.00	5750.00	115
39.40	2115.00	5750.00	115
0.00	3350.20	8000.00	125

Table 6.B-6

Geologic Log for Treasure Island (water table at 4 ft)

Depth to Top (ft)	Description
0	loose, gravelly SAND
2	loose, well-sorted, fine to medium SAND
5	well-sorted, very fine to fine SAND
28	well-sorted, fine to medium SAND
36	very fine, sandy LOAM
44	soft CLAY
95	SAND and SHELLS
103	SAND and LOAM, with shell fragments and some gravel pieces
121	CLAY
124	loamy, fine to very fine SAND
136	stiff CLAY
154	SAND
158	stiff CLAY
171	hard CLAY LOAM
204	SAND
207	stiff to very stiff CLAY
248	fine, gravelly SAND
266	CLAY
286	dense WOOD
289	hard, sandy CLAY
299	firm SHALE
310	greywacke SANDSTONE
314	SHALE
322	fine-grained SANDSTONE
340	bottom of hole

Table 6.B-7
Properties of Samples from Treasure Island

Sample No.	Sample Depth ft (m)	Soil Description	Liquid Limit %	Plasticity Index %	Water Content %	Total Unit Weight pcf (g/cc)	Void Ratio e	Dry Unit Weight pcf (g/cc)	Specific Gravity Gs	Degree of Saturation %
T1	17.5 (5.3)	Medium Fine Sand	----	----	19.9	120 (1.92)	0.66	100 (1.60)	2.65	80
T2	30 (9.1)	Dark Gray Silty Sand	----	----	21.3	120 (1.92)	0.67	99 (1.59)	2.65	85
T3	60 (18.3)	Dark Greenish Soft Clay	51	26	50.2	108 (1.73)	1.34	72 (1.15)	2.70	100
T4	90 (27.4)	Dark Gray Soft Clay	42	19	41.9	113 (1.81)	1.10	80 (1.28)	2.70	100
T5	130 (39.6)	Dark Greenish Med.Stiff Clay with Shells	37	23	37.0	114 (1.83)	1.02	83 (1.33)	2.70	98
T6	170 (51.8)	Dark Greenish Gray Stiff Clay	34	19	20.7	128 (2.05)	0.58	106 (1.70)	2.70	96
T7	232 (70.7)	Dark Greenish Gray Clay w/Horiz.Beddings	48	30	33.3	115 (1.84)	0.95	87 (1.39)	2.70	95

Table 6.B-8

Modulus-Reduction and Damping Curves for Treasure Island

Strain (%)	G/GMAX (0-44 ft)	G/GMAX (44 + ft)
1.0E-4.0	1.00	1.00
1.0E-3.5	1.00	1.00
1.0E-3.0	0.99	1.00
1.0E-2.5	0.96	0.99
1.0E-2.0	0.85	0.95
1.0E-1.5	0.64	0.85
1.0E-1.0	0.34	0.63
1.0E-0.5	0.15	0.38
1.0E-0.0	0.06	0.18

Strain (%)	Damping % (0-44 ft)	Damping % (44 + ft)
1.0E-4.0	1.0	0.9
1.0E-3.5	1.0	0.9
1.0E-3.0	1.2	1.0
1.0E-2.5	1.7	1.1
1.0E-2.0	3.1	1.6
1.0E-1.5	6.0	2.7
1.0E-1.0	12.4	5.4
1.0E-0.5	19.2	9.7
1.0E-0.0	22.8	13.5

Table 6.B-9

Base-Case Profile for Lotung LSST

Thickness (ft)	Vs (ft/s)	Vp (ft/s)	Total Limit Weight (psf)
7.74	373.15	1312.40	112
4.92	396.28	1935.79	112
0.89	396.28	2378.73	112
4.49	490.21	2378.73	112
1.64	490.21	3379.43	112
2.23	490.21	3379.43	112
4.13	540.41	3379.43	115
3.35	540.41	4691.83	115
6.69	590.58	4691.83	115
0.66	590.58	4691.83	112
9.91	607.31	4691.83	112
9.12	621.45	4691.83	112
0.95	621.45	4691.83	125
8.37	771.99	4691.83	125
7.42	800.30	4691.83	125
5.22	800.30	4330.92	119
5.94	726.97	4330.92	119
12.76	747.54	4527.78	119
1.35	1047.36	4527.78	119
13.35	1047.36	4675.75	119
6.33	813.16	4675.75	119
3.74	813.16	4363.73	119
6.82	850.50	4363.73	112
12.50	823.47	4363.73	112
13.68	840.20	4363.73	125
8.24	840.20	4363.73	125

Table 6.B-10

Geologic Log* for Lotung LSST (water table at 0 ft)

Depth to Top (ft)	Description
0	fill:clayey SILT with gravel
5	very loose, silty fine SAND, with some beds of clayey silt and decayed wood
27	clayey to sandy SILT, soft
35	loose silty fine SAND, with some beds of silty clay and sandy silt
43	medium dense GRAVEL
48	decayed WOOD and organic matter with clayey silt
61	silty fine SAND
71	silty CLAY to clayey SILT, stiff
78	silty fine to medium SAND with gravel, medium dense
81	silty fine to medium SAND with some medium to coarse sand and gravel
93	silty SAND and sandy SILT
102	sandy GRAVEL, dense
108	silty CLAY to clayey SILT, very stiff, with some beds of decayed wood and silty sand
148	clayey and sandy SILT
154	bottom of profile

*From hole CH-2, and last layer from hole L4, Anderson and Tang (1989).

Table 6.B-11

Properties of Samples from Lotung LSST

Sample No.	Depth ft (m)	Soil Description	Liquid Limit %	Plasticity Index %	Water Content %	Total Unit Weight pcf (g/cc)	Void Ratio e	Specific Gravity Gs	Degree of Saturation %
CH1(T1)	18 (5.5)	Silt	-----	-----	31.0	112 (1.79)	0.93	2.65	88
CH2(T5)	34.5 (10.5)	Silt	-----	-----	32.5	118 (1.89)	0.85	2.65	100
CH1(T4)	59 (18.0)	Silty Fine Sand	-----	-----	33.3	109 (1.75)	1.02	2.65	87
CH2(T9)	93.5 (28.5)	Silty Sand	-----	-----	31.2	119 (1.91)	0.82	2.65	100
CH1(T8)	113 (34.5)	Clayey Silt	32	7	35.3	118 (1.89)	0.92	2.70	100
CH2(T11)	133 (40.5)	Clayey Silt	33	8	31.1	117 (1.88)	0.89	2.70	95
CH1(T10)	146 (44.5)	Silt	-----	-----	24.0	128 (2.05)	0.56	2.65	100

Table 6.B-12

Modulus-Reduction and Damping Curves for Lotung LSST

Strain (%)	G/GMAX (0-154 ft)
1.0E-4.0	1.00
1.0E-3.5	1.00
1.0E-3.0	1.00
1.0E-2.5	0.97
1.0E-2.0	0.87
1.0E-1.5	0.69
1.0E-1.0	0.42
1.0E-0.5	0.21
1.0E-0.0	0.09

Strain (%)	Damping% (0-154ft)
1.0E-4.0	1.0
1.0E-3.5	1.0
1.0E-3.0	1.1
1.0E-2.5	1.5
1.0E-2.0	2.8
1.0E-1.5	5.3
1.0E-1.0	10.5
1.0E-0.5	16.8
1.0E-0.0	22.1

Table 6.B-13

Control Motions

Earthquake	M	Site	Distance(km)	PGA (g)	
				NS	EW
1989 Loma Prieta	6.9	Gilroy 1	12	0.411	0.473
		Yerba Buena Is.	77	0.029	0.068
LSST Event 7 05/20/86	6.5	LSST-DB47 (154 ft depth)	64	0.099	0.081
LSST Event 16 11/14/86	7.8	LSST-DB17 (56 ft depth)	79	0.086	0.074
LSST Event 10 07/16/89	4.5	LSST-DB47 (154 ft depth)	6	0.030	0.040

Table 6.B-14
Frequency And Amplitude Of Fundamental Resonances

Surface/20 ft			
LSST Event	Surface AVG PGA (g)	F (Hz)	Amplitude
10	0.035	4.83	10.84
16	0.080	3.37	3.78
7	0.090	2.98	2.65
Surface/36 ft			
10	0.035	3.37	7.16
16	0.080	2.44	3.38
7	0.090	1.71	2.01
Surface/56 ft			
10	0.035	2.44	8.15
16	0.080	1.66	3.68
7	0.090	1.32	2.52
Surface/154 ft			
10	0.035	1.22	6.57
16	0.080	-----*	-----*
7	0.090	0.78	3.58

*Recording not available

Table 6.B-15
Magnitudes (M) and Distances used in Outcrop Motions

Ap(g)	f _p *(Hz)	V _p ** (cm/sec)	f _p *(Hz)	V/A(cm/sec/g)	M	R(km)
0.50	5.21	57.12	1.15	114.24	7.0	11.49
0.75	5.20	97.56	1.02	130.08	7.5	10.89
1.00	5.21	136.40	0.98	136.40	8.0	10.08
1.25	5.26	168.70	0.98	135.00	8.0	8.22

*Predominant frequencies associated with peak acceleration and peak particle velocity estimated by random process theory.

**V_p Peak particle velocity

Outcrop motions are computed assuming a halfspace with a source region shear-wave velocity and density of 3.2 km/sec and 2.7 cgs, a stress drop of 100 bars, a $Q(f) = 150 f^{0.6}$, and a Wald et al., (1991) crustal model.

Table 6.B-16

Comparison of Run Times*

Code	Run Time (sec)**
DESRA	96
RASCAL	132
SUMDES	712
TESS	62

*NS component for the Loma Prieta earthquake at Gilroy 2.

**PC compatible 50 MHz 486.

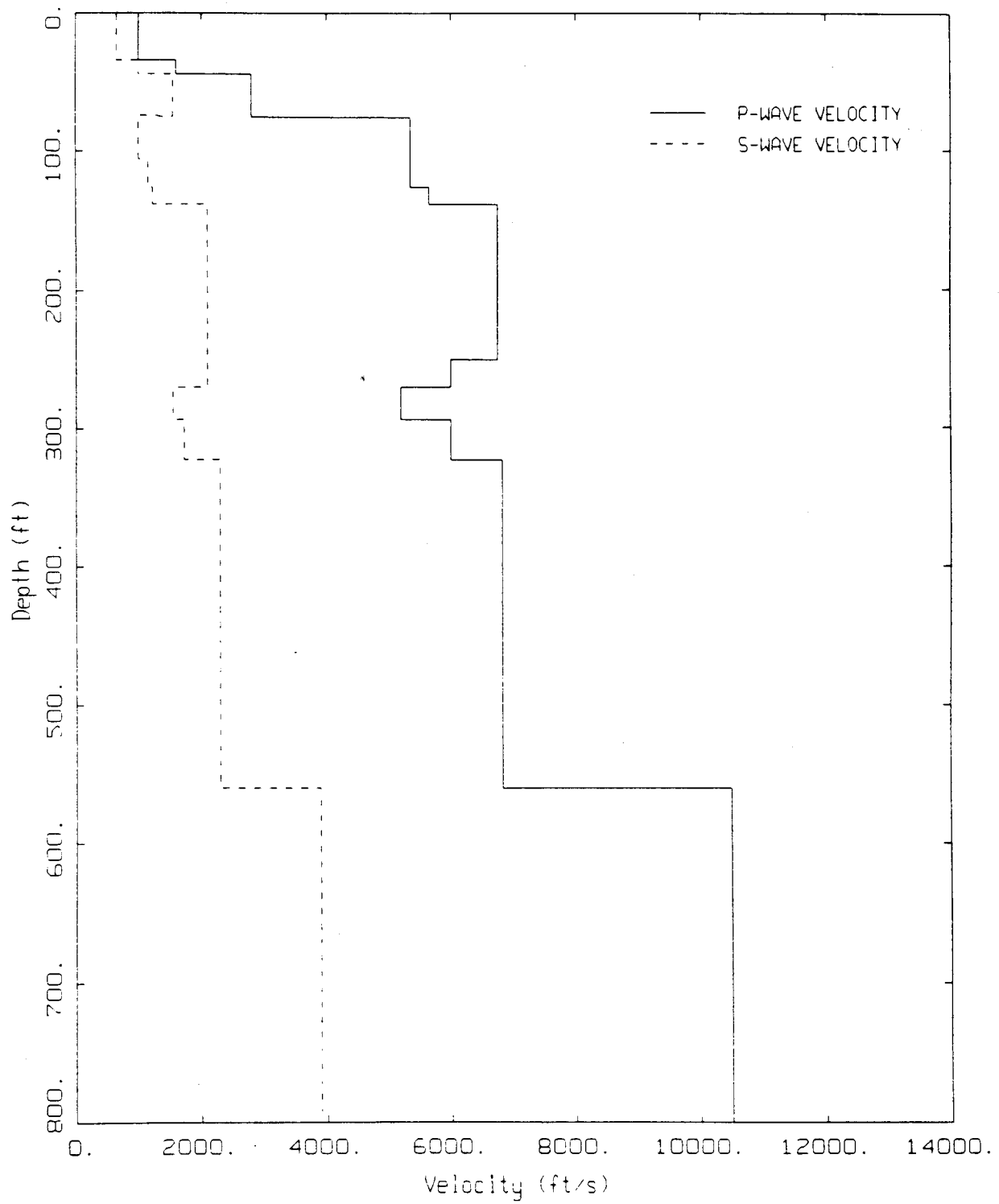


Figure 6.B-1
Base-case shear- and compressional-wave velocities for reference site Gilroy 2.

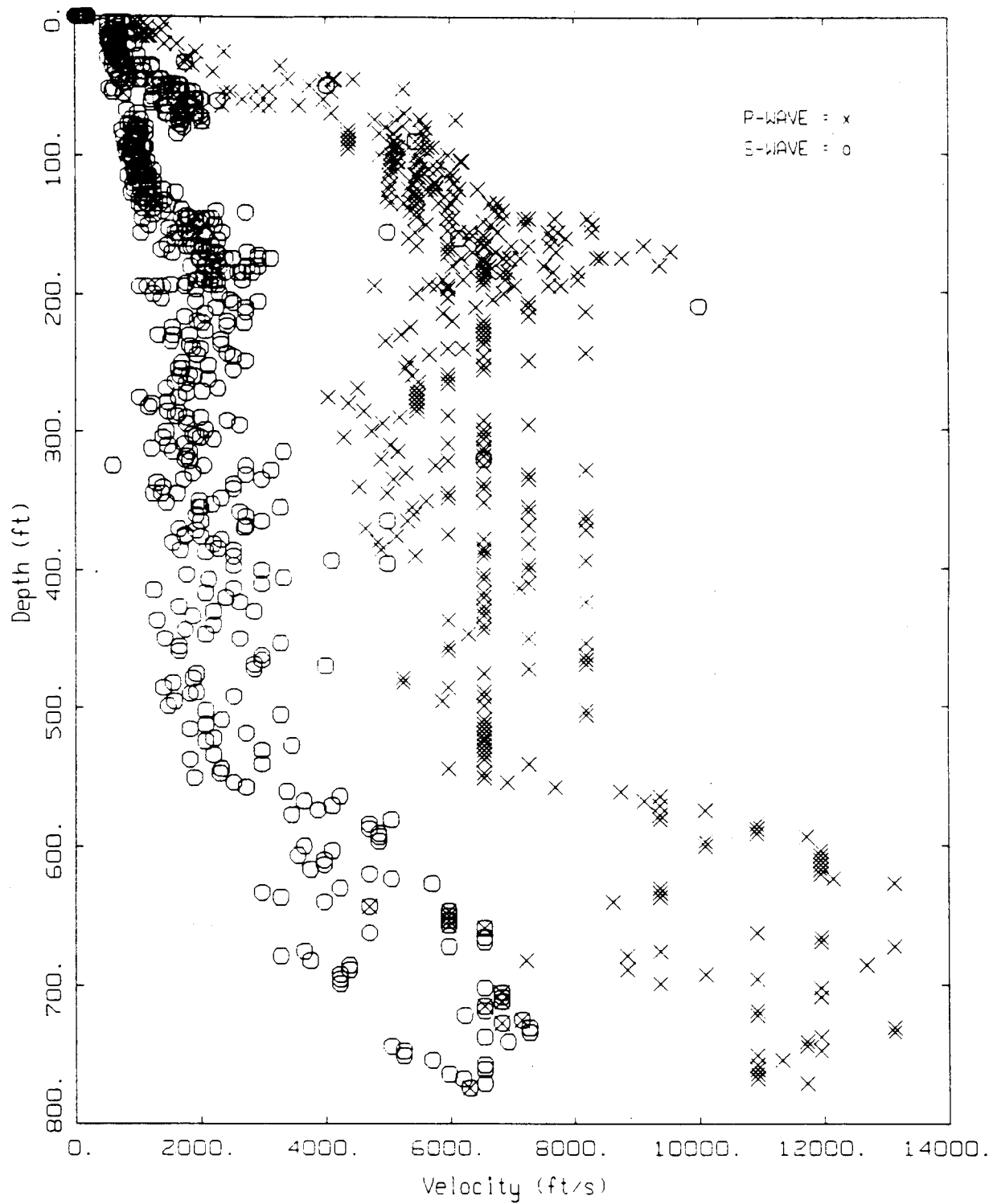


Figure 6.B-2

In-situ shear- and compressional-wave velocities for reference site Gilroy 2. Measurement techniques represent downhole, crosshole, and suspension log.

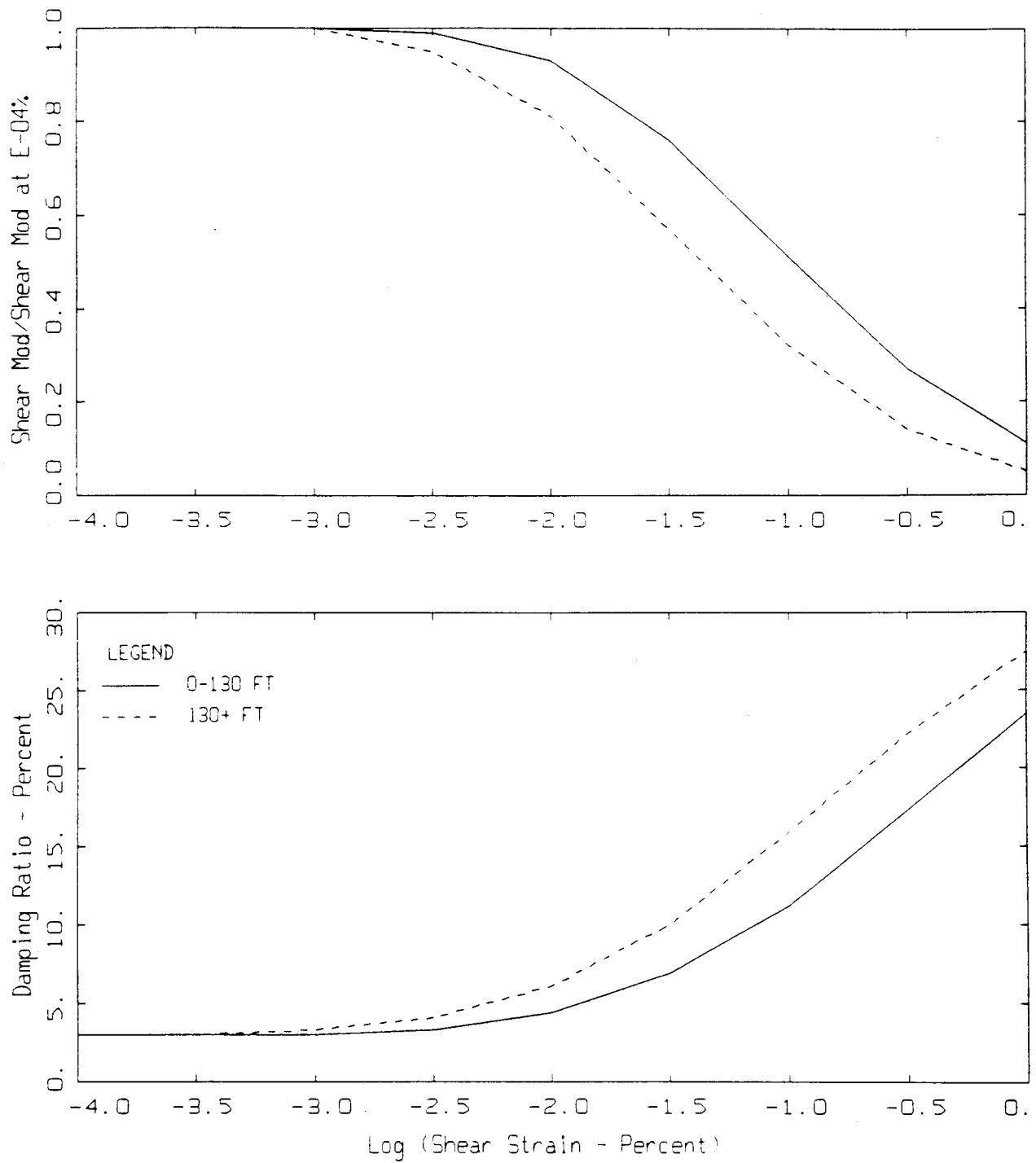


Figure 6.B-3

Modulus reduction and damping curves for reference site Gilroy 2. The curves are based on laboratory testing of "undisturbed" samples.

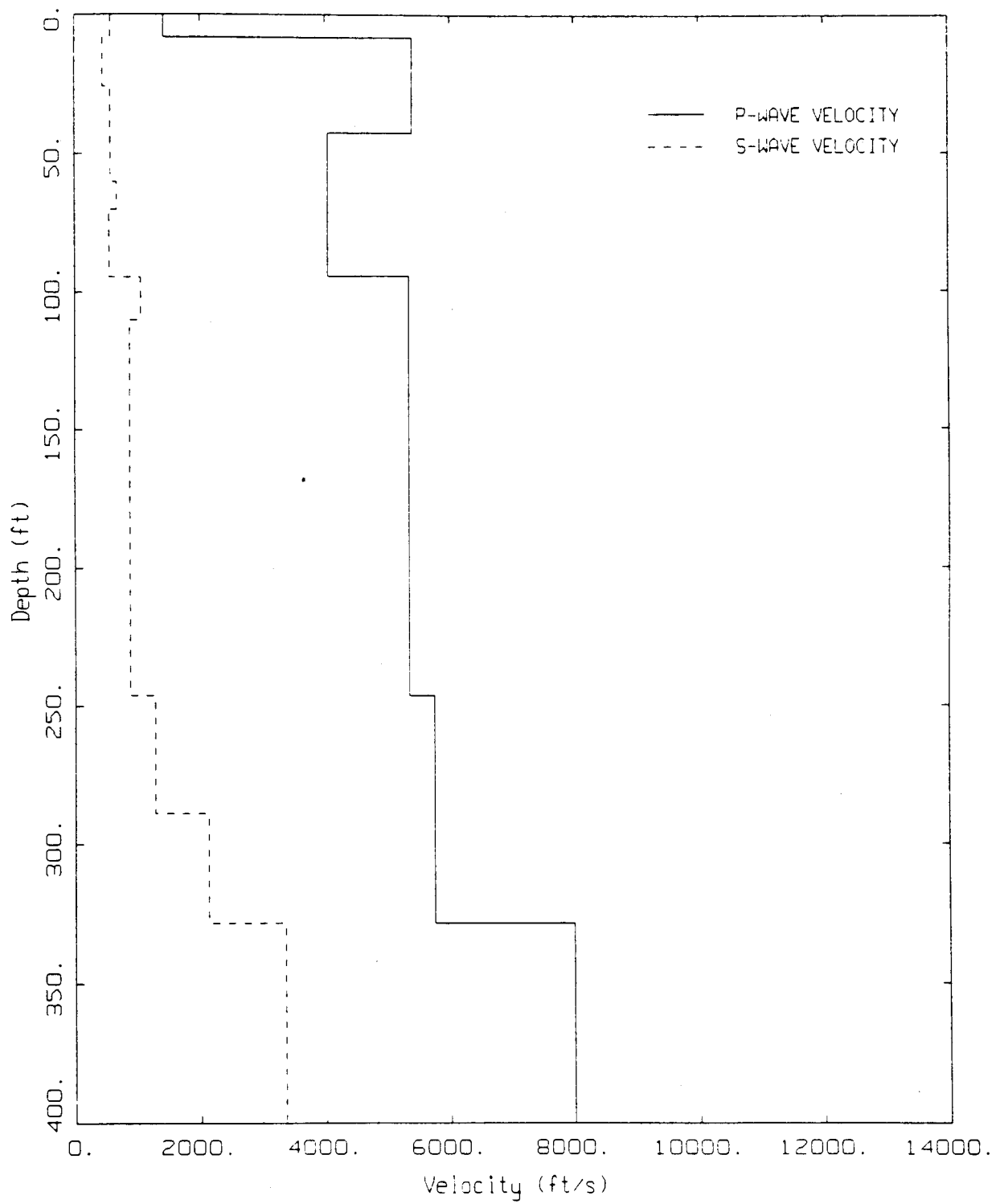


Figure 6.B-4
Base-case shear- and compressional-wave velocities for reference site Treasure Island.

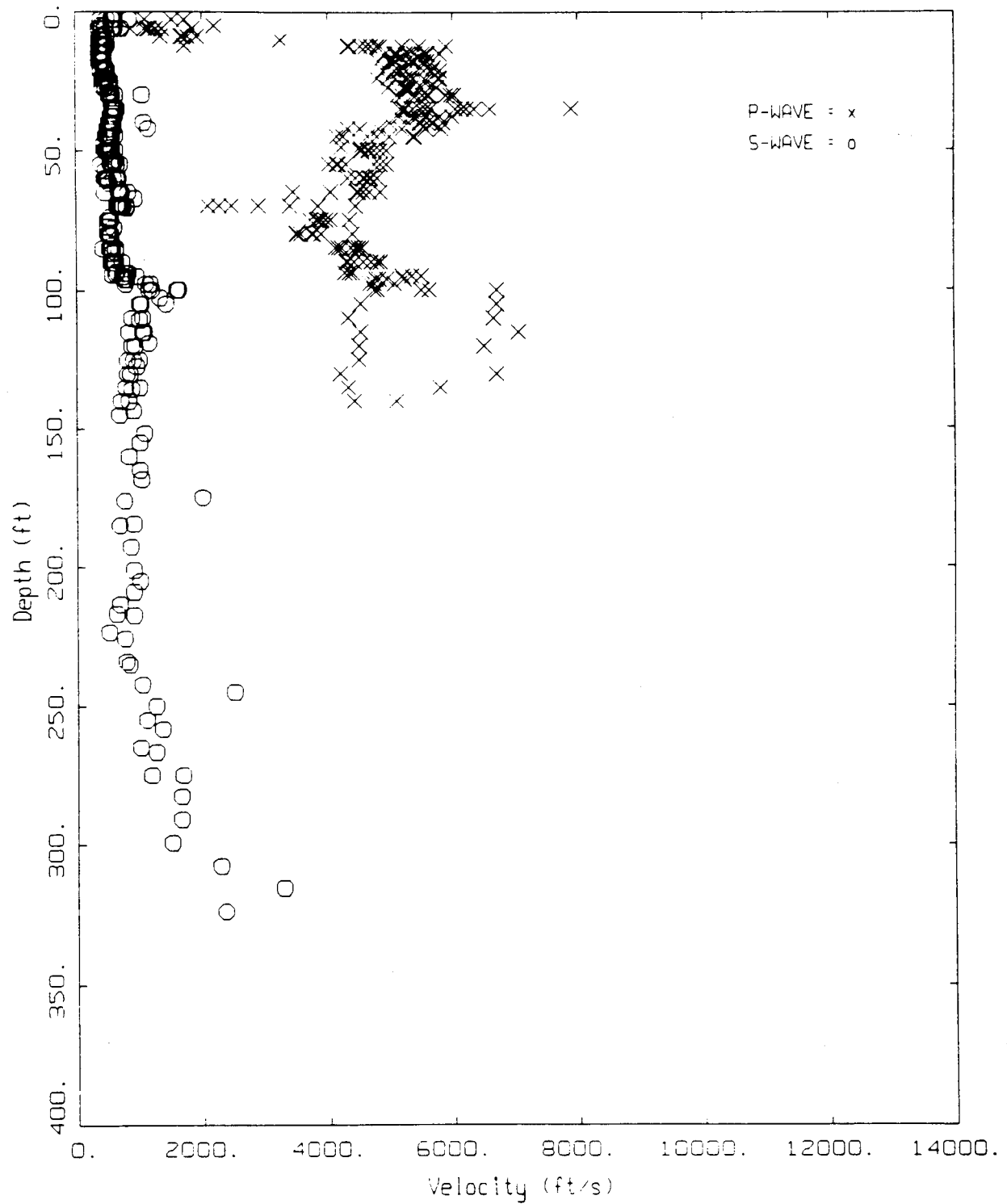


Figure 6.B-5
In-situ shear- and compressional-wave velocities for reference site Treasure Island. Measurement techniques represent downhole, crosshole, and suspension log.

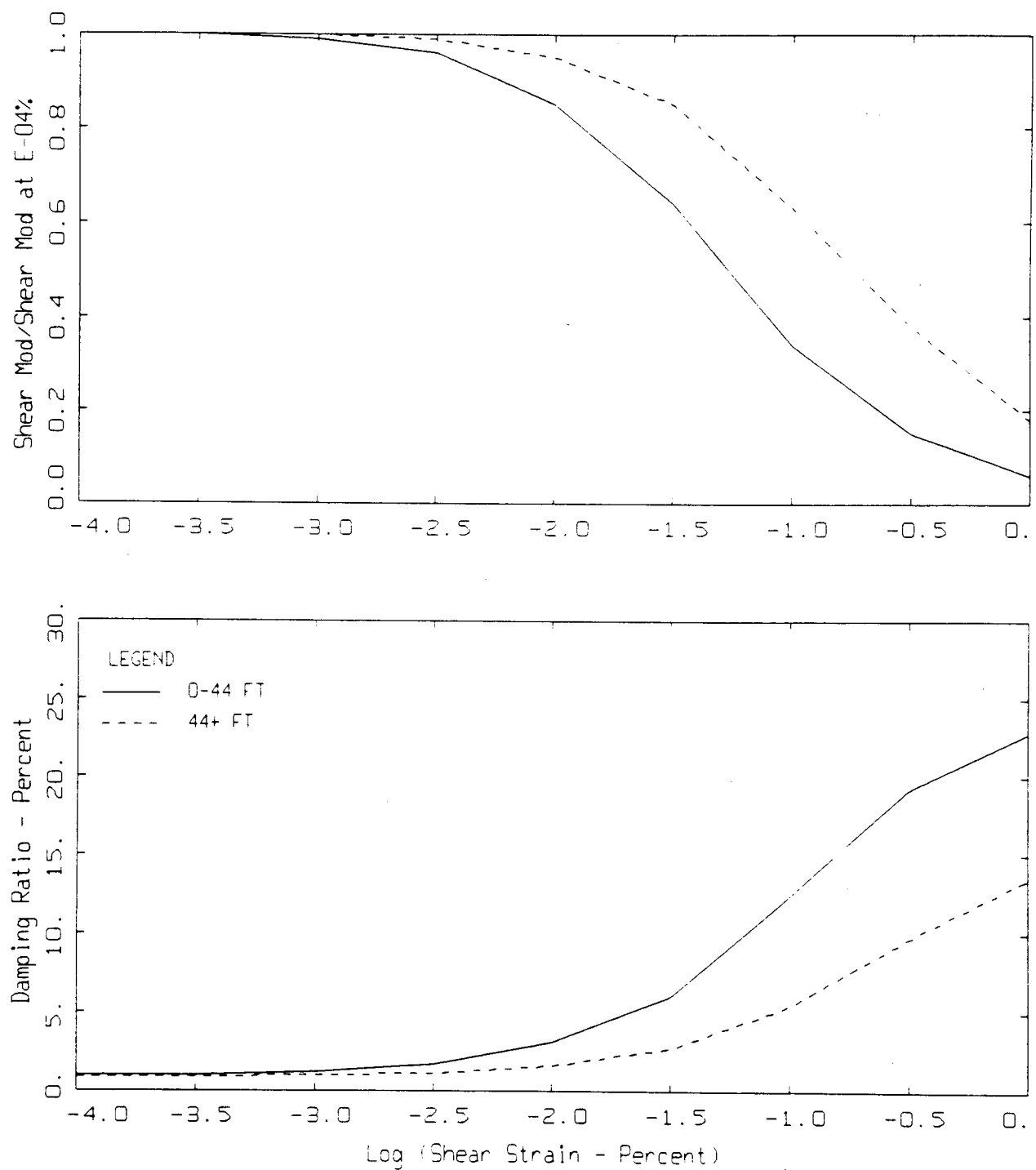


Figure 6.B-6

Modulus reduction and damping curves for reference site Treasure Island. The curves are based on laboratory testing of "undisturbed" samples.

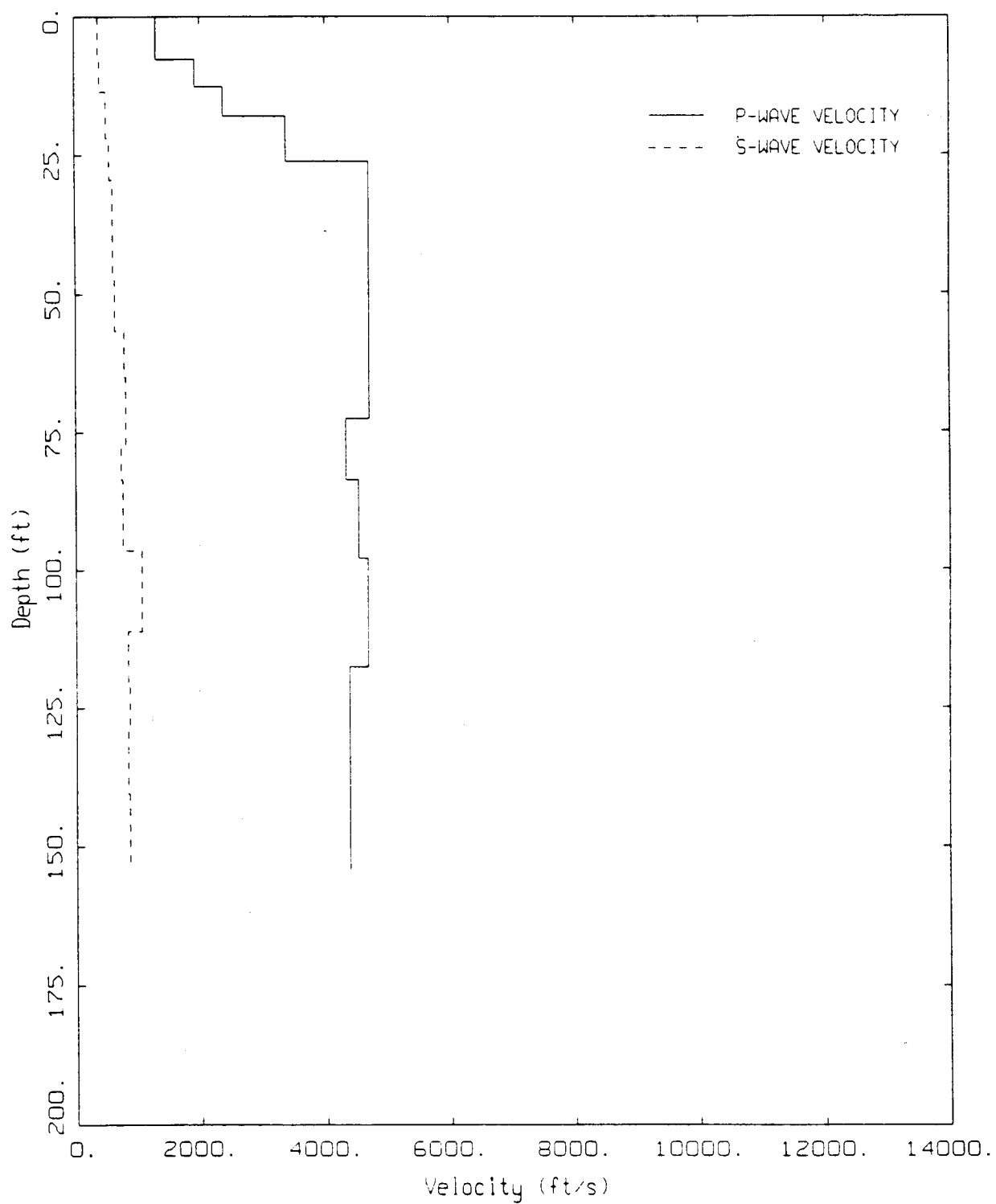


Figure 6.B-7
Base-case shear- and compressional-wave velocities for reference site Lotung LSST.

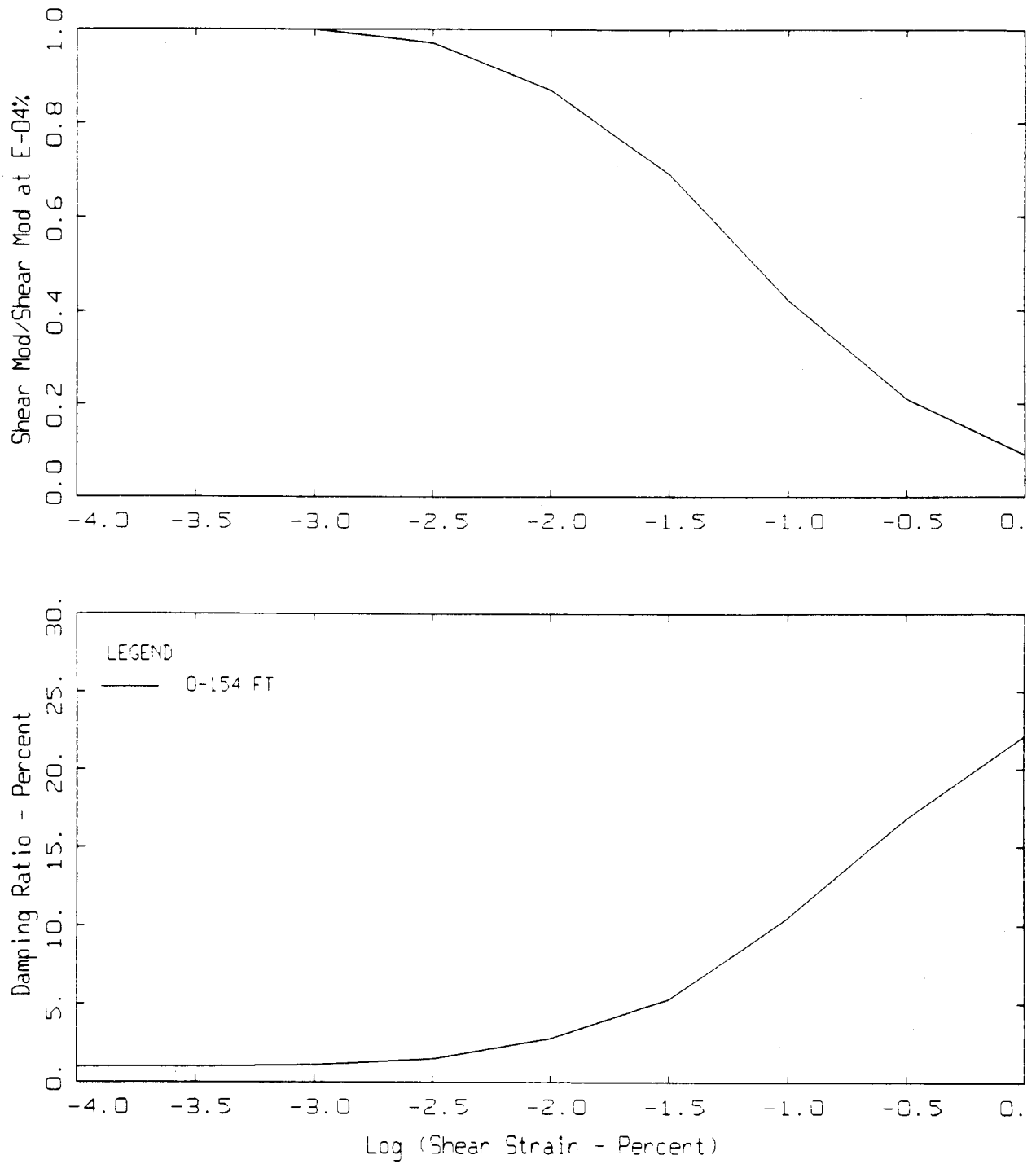


Figure 6.B-8

Modulus reduction and damping curves for reference site Lotung LSST. The curves are based on laboratory testing of "undisturbed" samples.

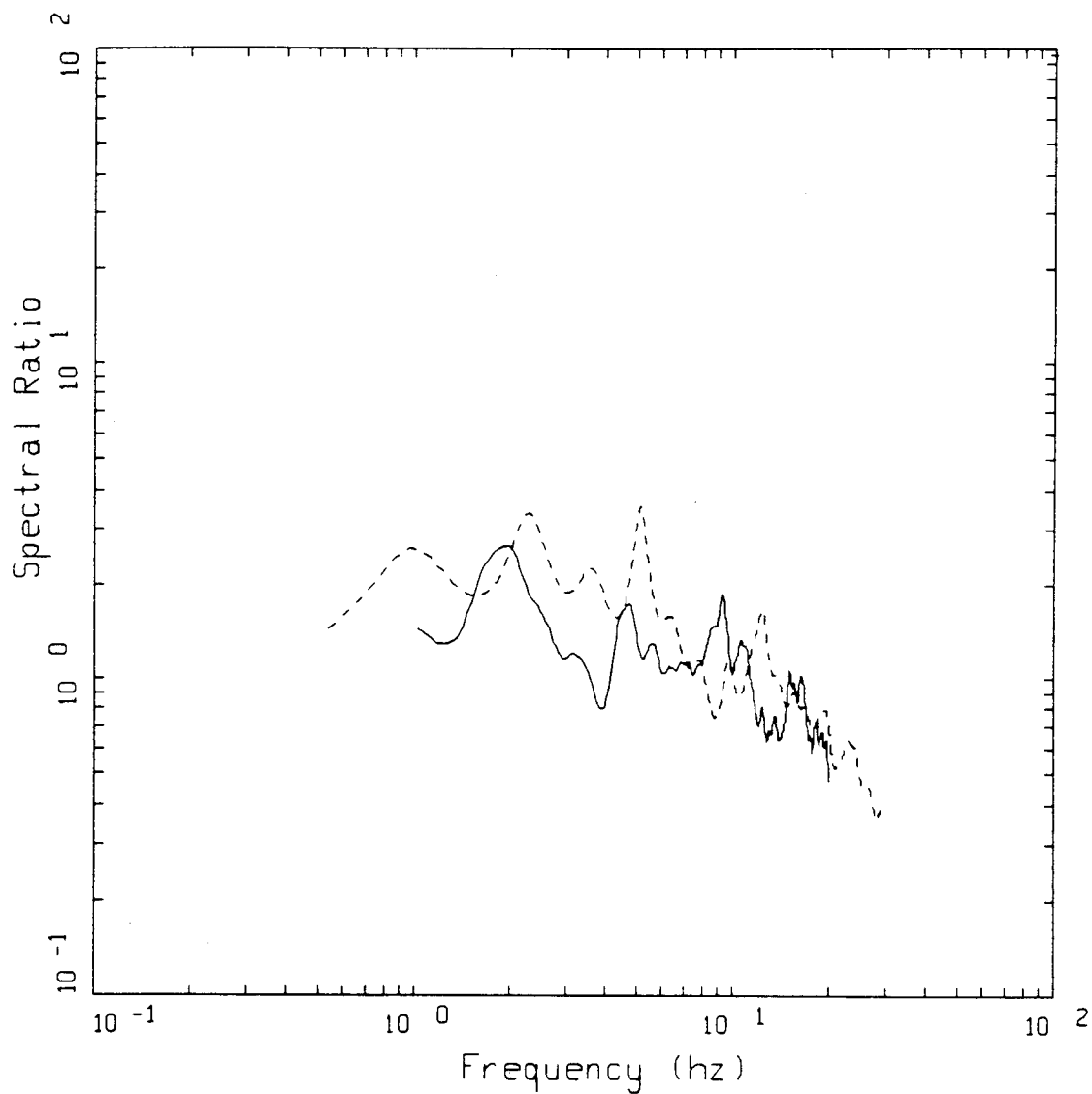


Figure 6.B-9

Fourier amplitude spectral ratios at reference site Gilroy 2 based on average of 13 aftershocks (solid) and computed using base-case shear-wave velocity profile (dashed). Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

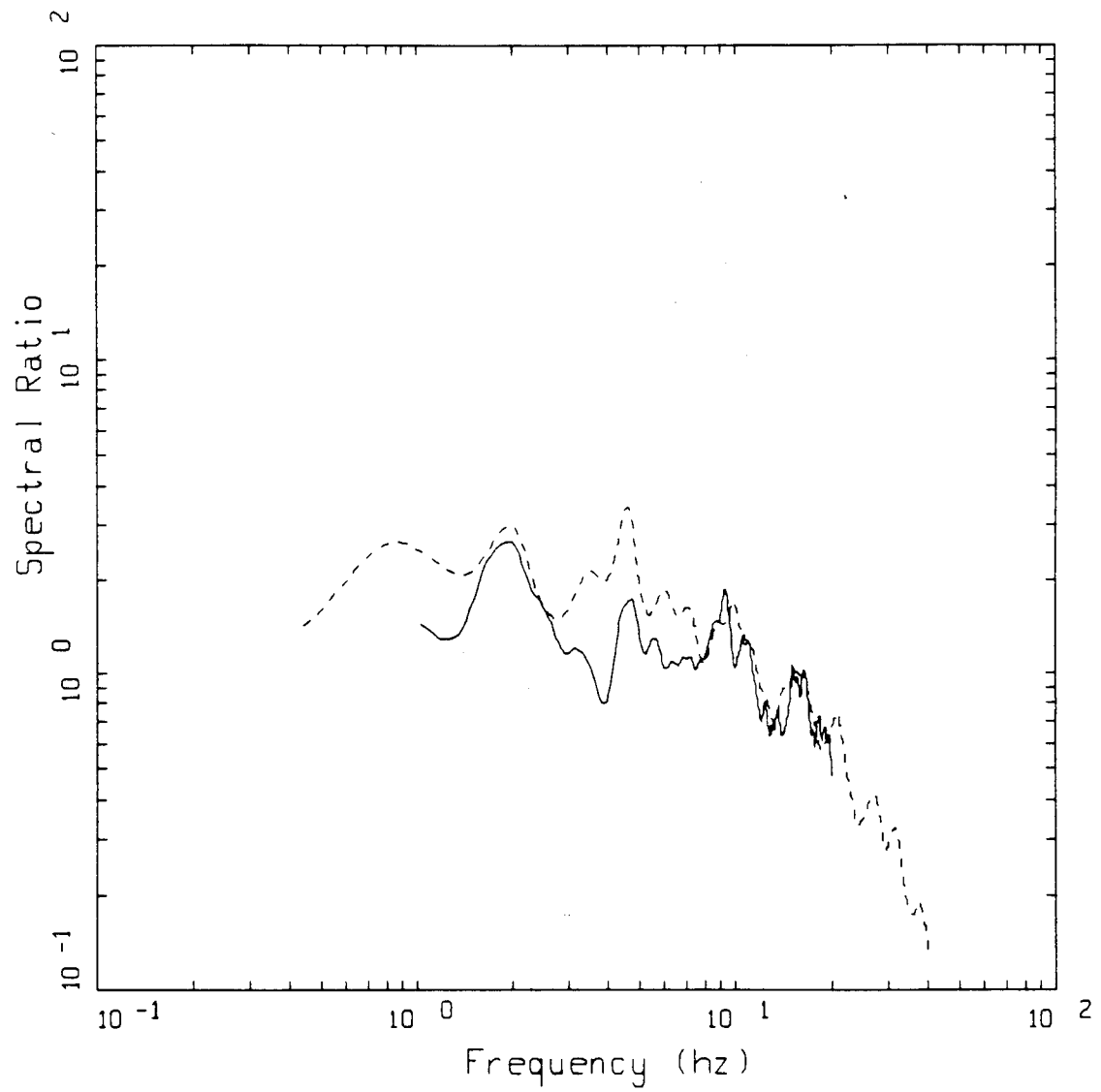


Figure 6.B-10

Fourier amplitude spectral ratios at reference site Gilroy 2 based on average of 13 aftershocks (solid) and computed using revised (Figure 6.B-11) shear-wave velocity profile (dashed). Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

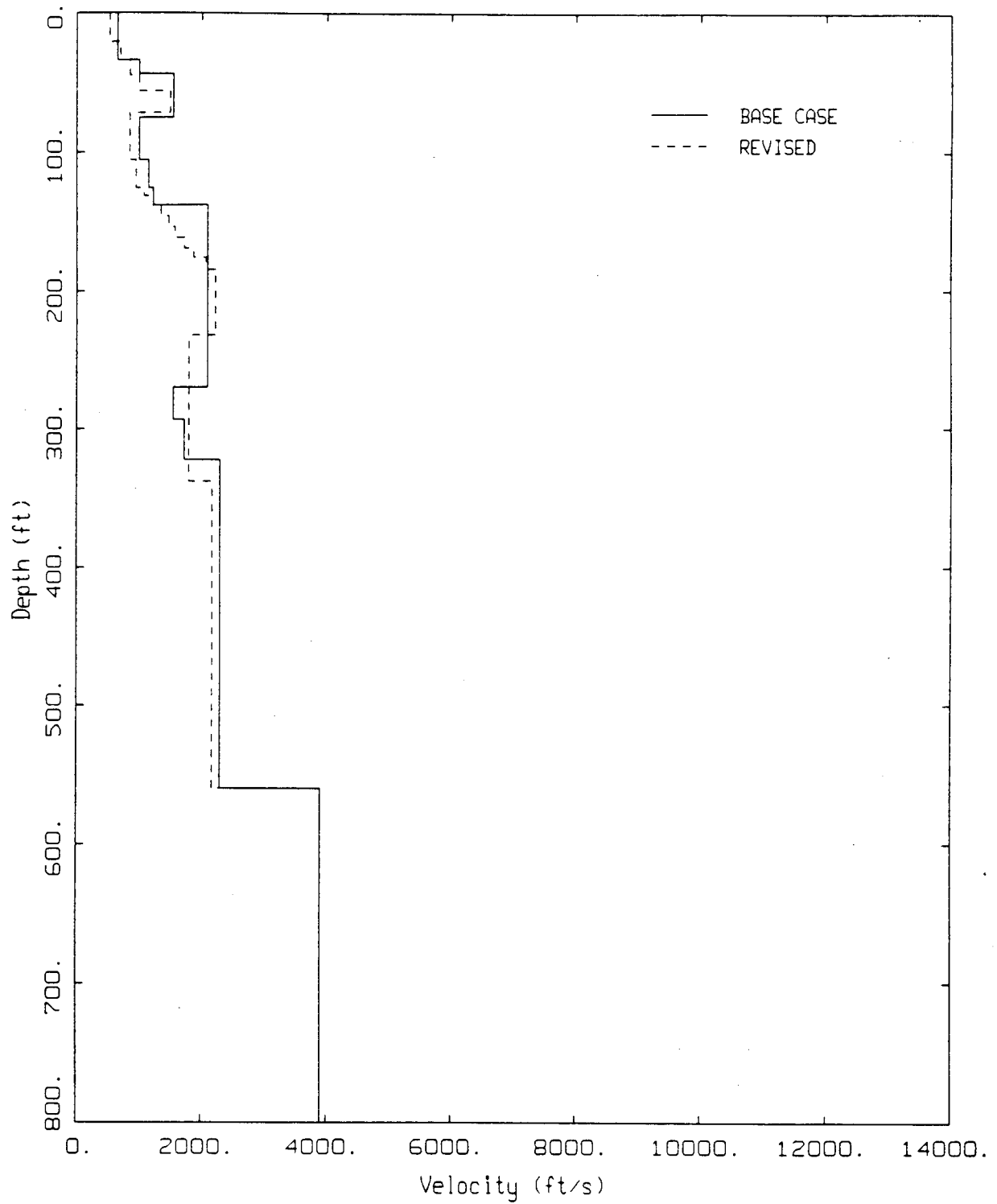


Figure 6.B-11
Base-case and revised shear-wave velocities for reference site Gilroy 2.

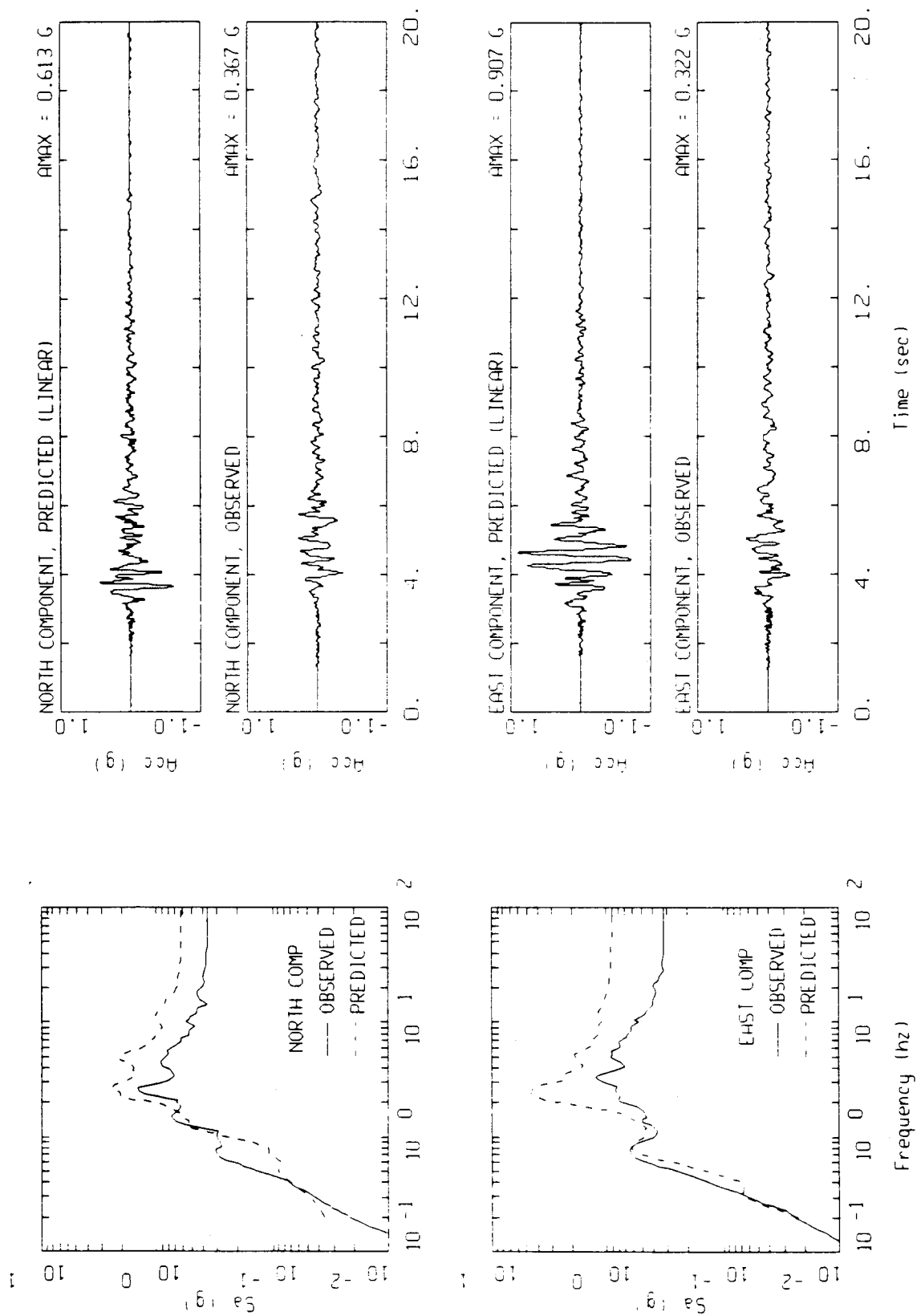


Figure 6.B-12

Comparison of observed and predicted time histories and 5% damped spectral accelerations for the Loma Prieta earthquake at reference site Gilroy 2 using Gilroy 1 rock outcrop as control motions. RASCAL linear analysis.

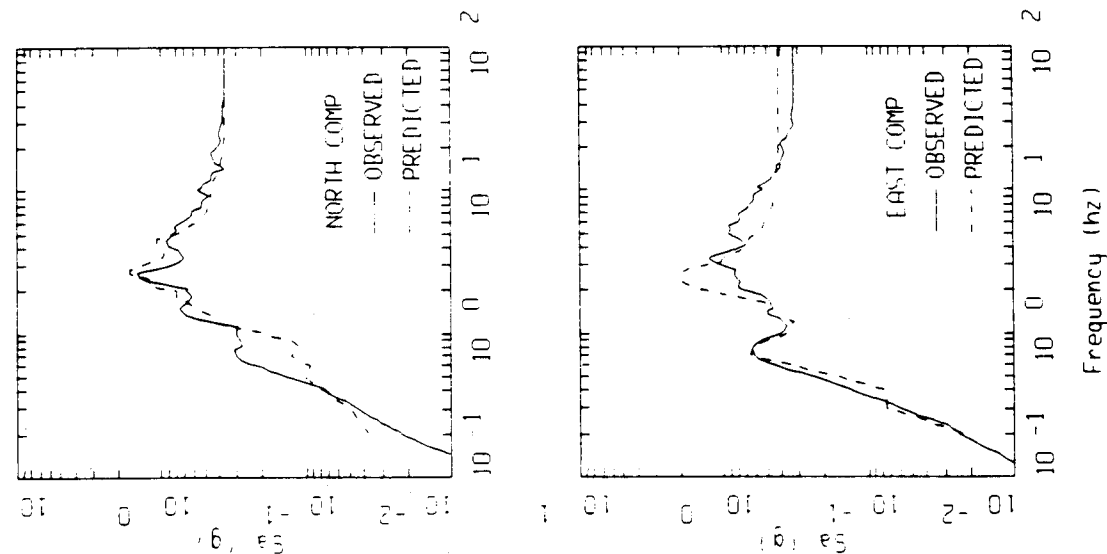
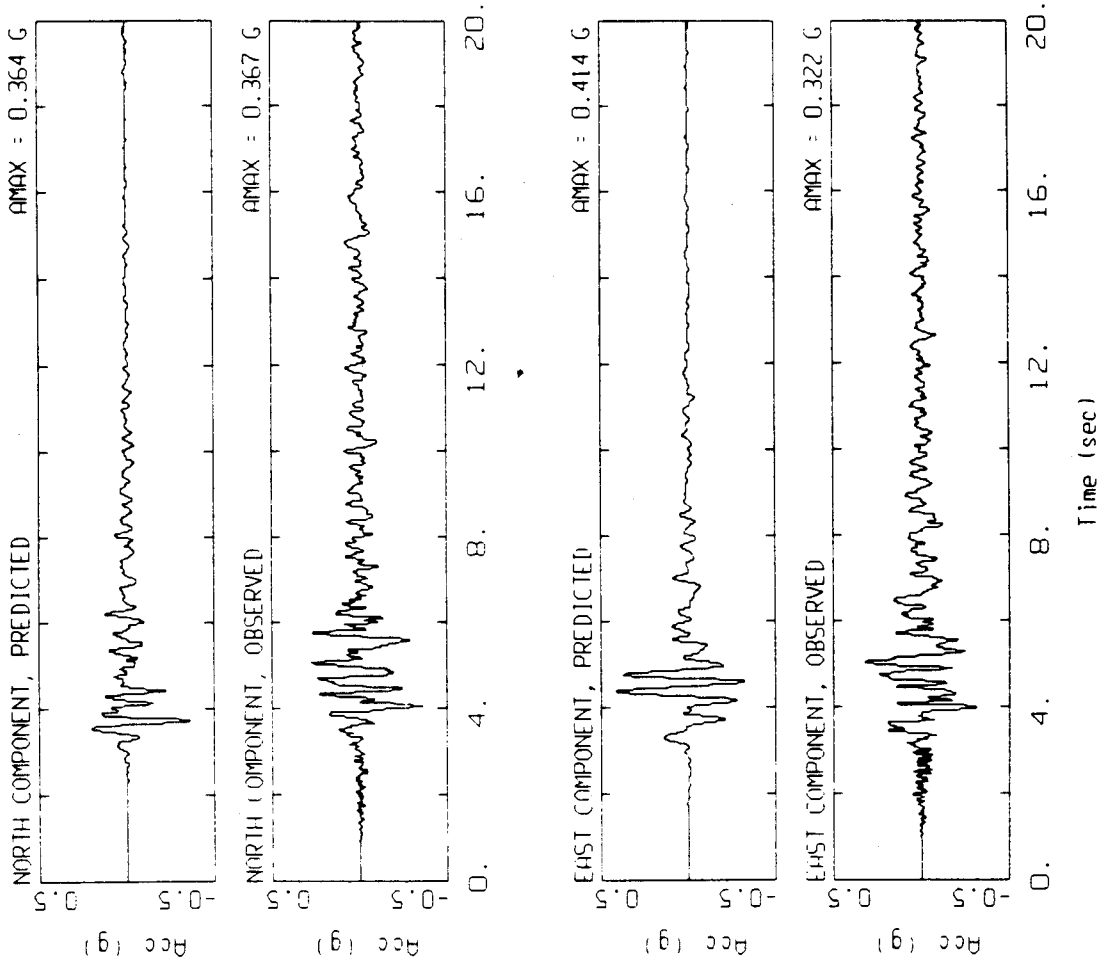


Figure 6.B-13
Comparison of observed and predicted time histories and 5% damped spectral accelerations for the Loma Prieta earthquake at reference site Gilroy 2 using Gilroy 1 rock outcrop as control motions. RASCAL equivalent-linear analysis.

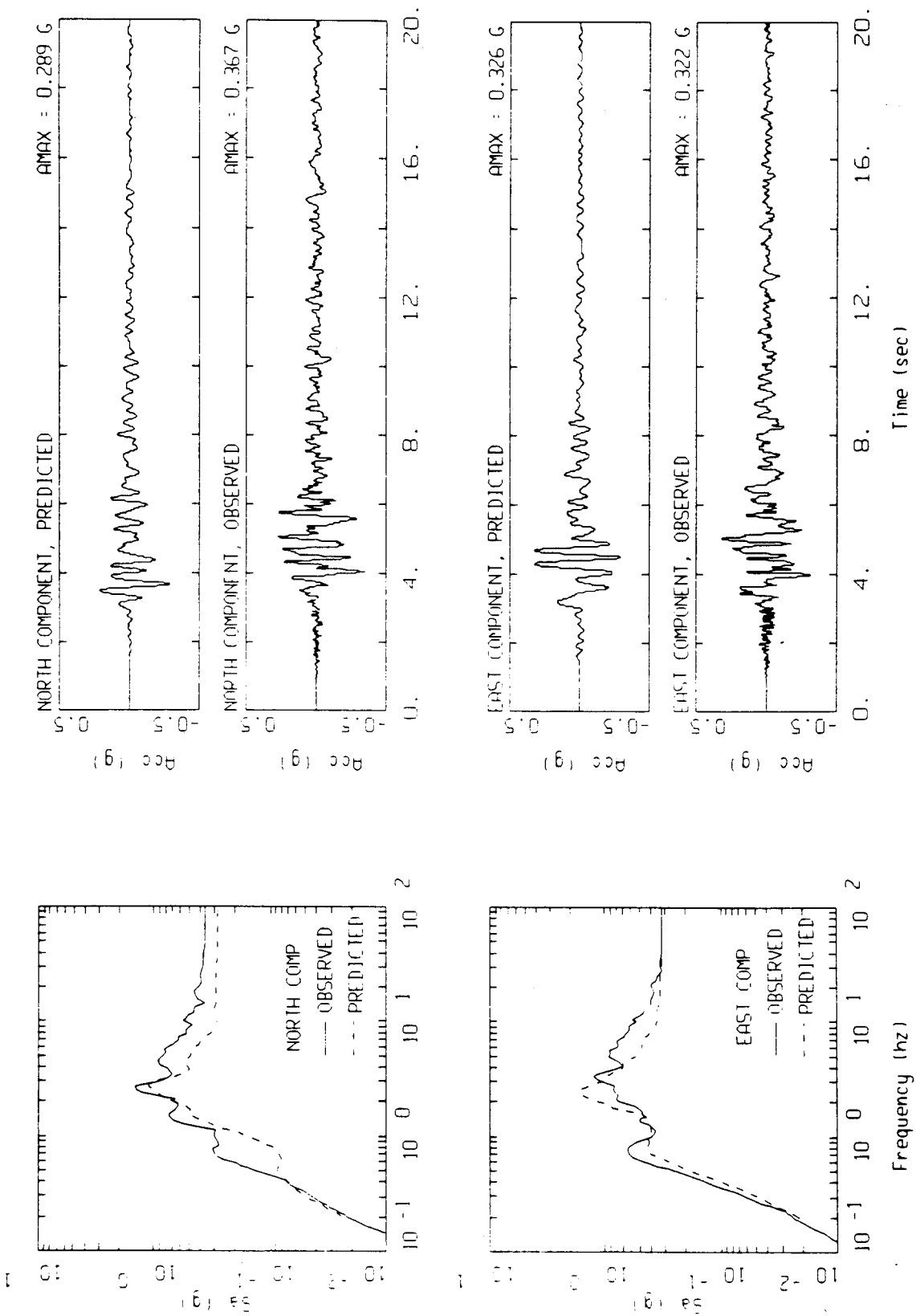


Figure 6.B-14

Comparison of observed and predicted time histories and 5% damped spectral accelerations for the Loma Prieta earthquake at reference site Gilroy 2 using Gilroy 1 rock outcrop as control motions. DESRA analysis.

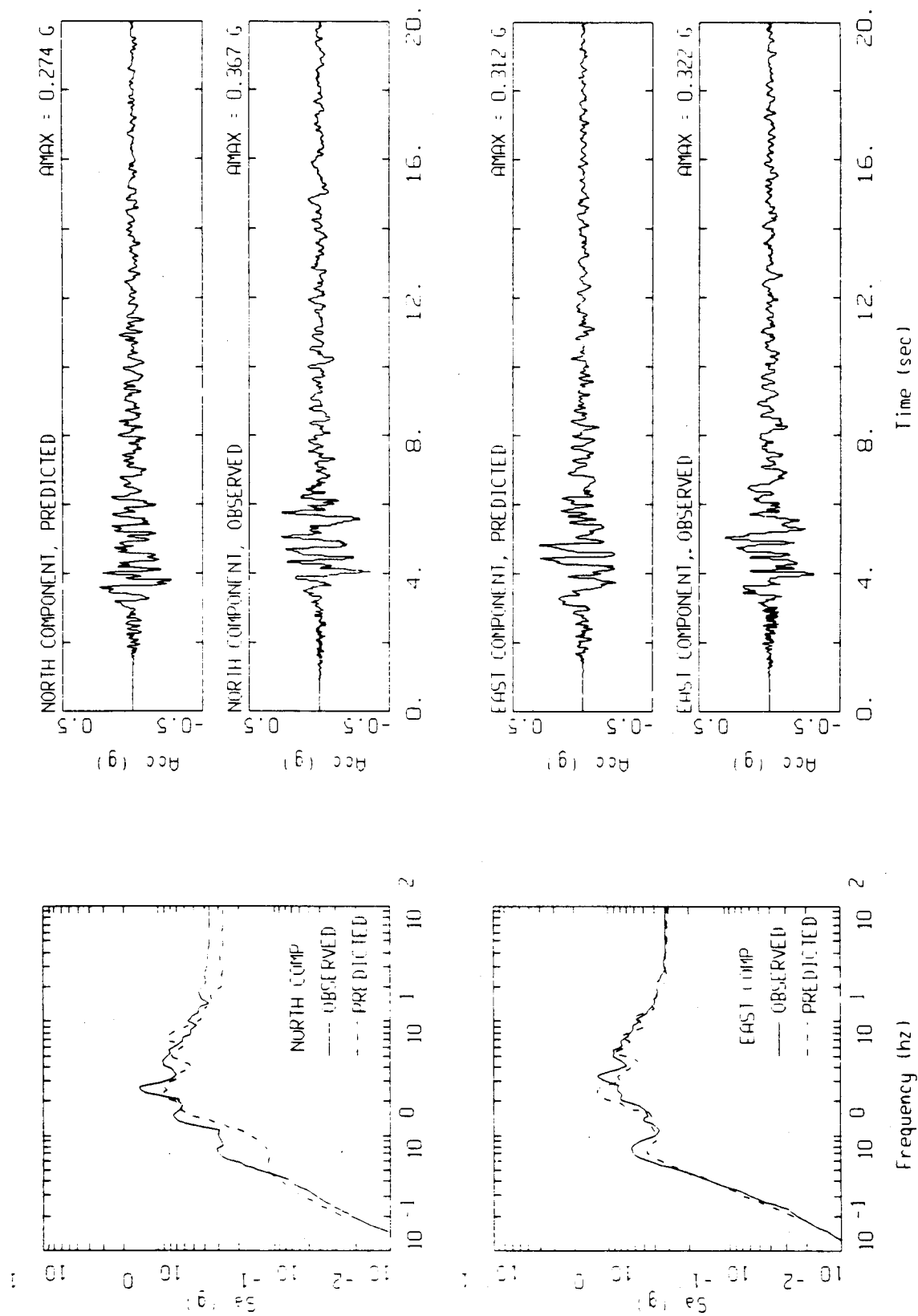


Figure 6.B-15
Comparison of observed and predicted time histories and 5% damped spectral accelerations for the Loma Prieta earthquake at reference site Gilroy 2 using Gilroy 1 rock outcrop as control motions. SUMDES analysis.

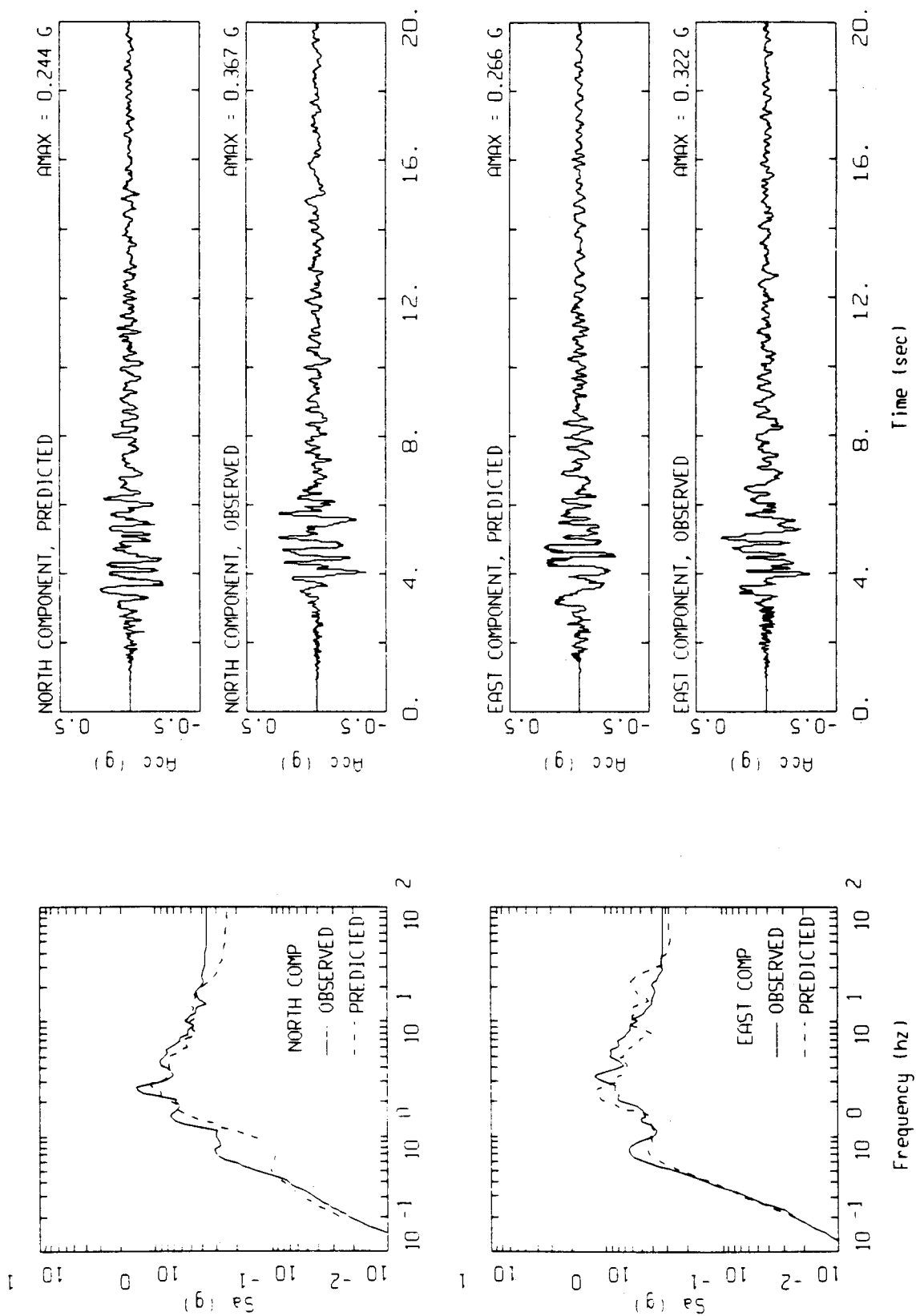


Figure 6.B-16

Comparison of observed and predicted time histories and 5% damped spectral accelerations for the Loma Prieta earthquake at reference site Gilroy 2 using Gilroy 1 rock outcrop as control motions. TESS analysis.

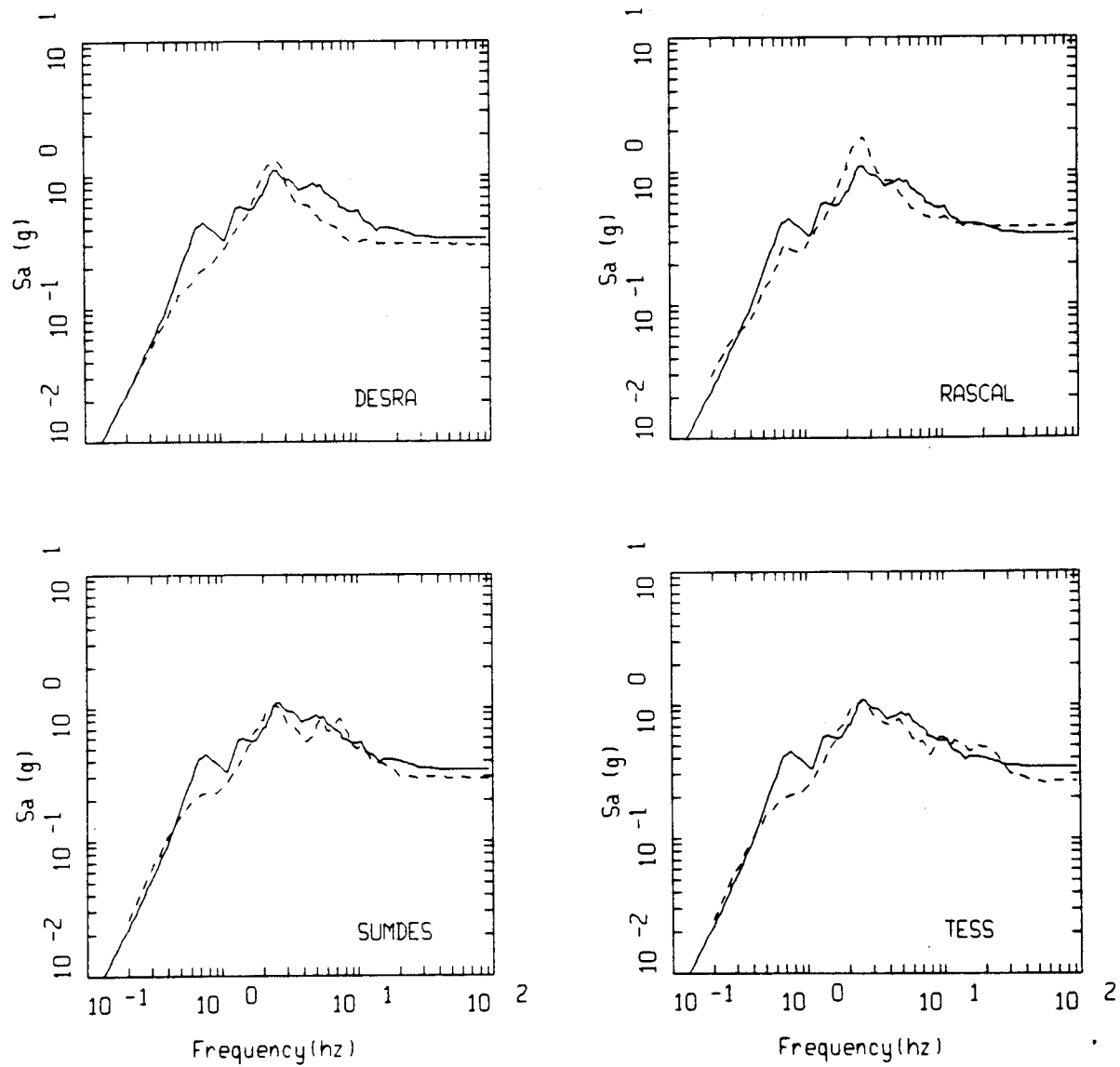


Figure 6.B-17

Comparison of log average (horizontal components) 5% damped spectral accelerations for the four analyses to the empirical spectra at reference site Gilroy 2 for the Loma Prieta earthquake (empirical, solid; predicted, dashed).

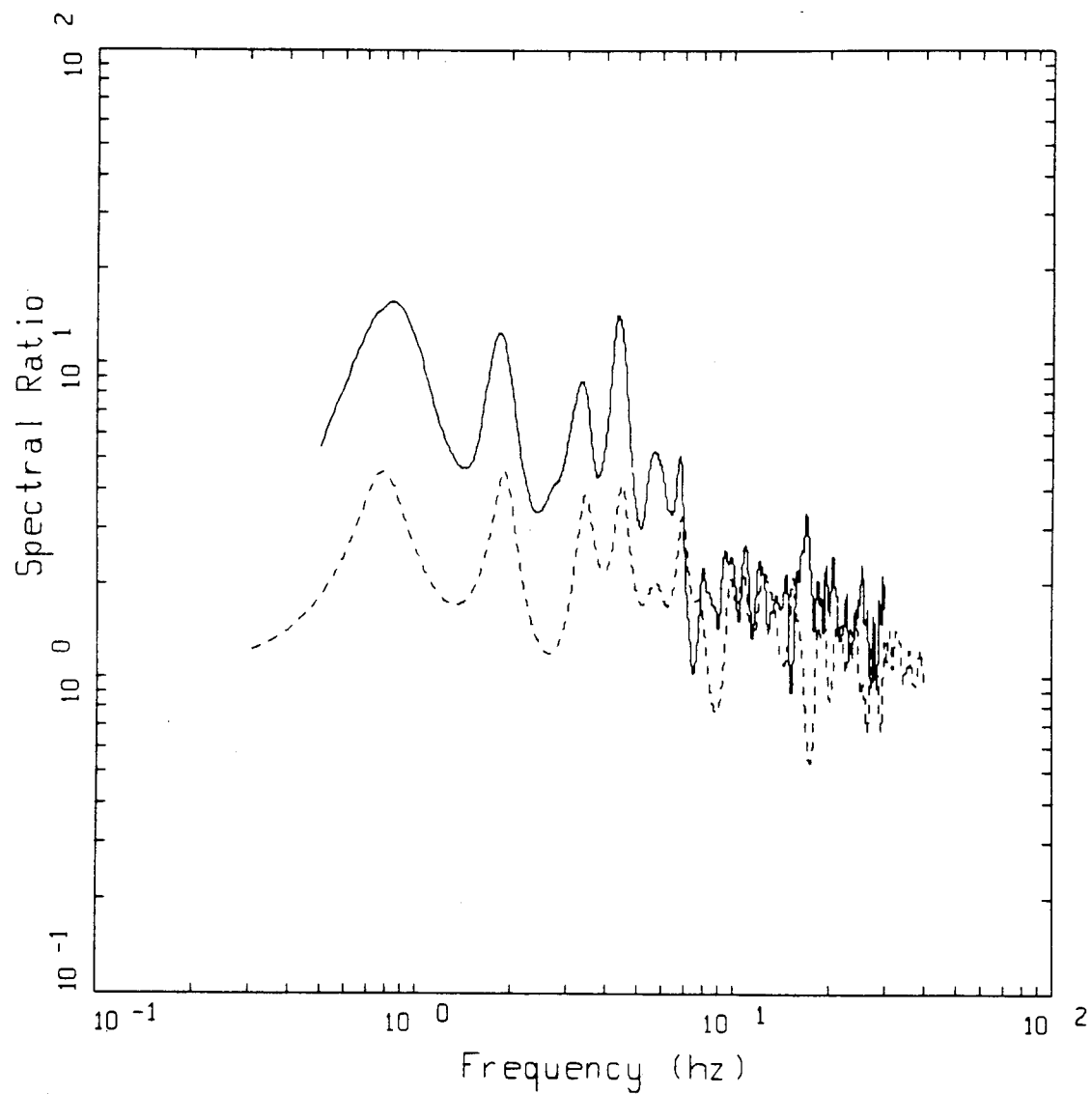


Figure 6.B-18

Fourier amplitude spectral ratios at reference site Treasure Island based on average of 7 aftershocks (solid) (Jarpe et al., 1989) and computed using base-case shear-wave velocity profile (dashed). Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

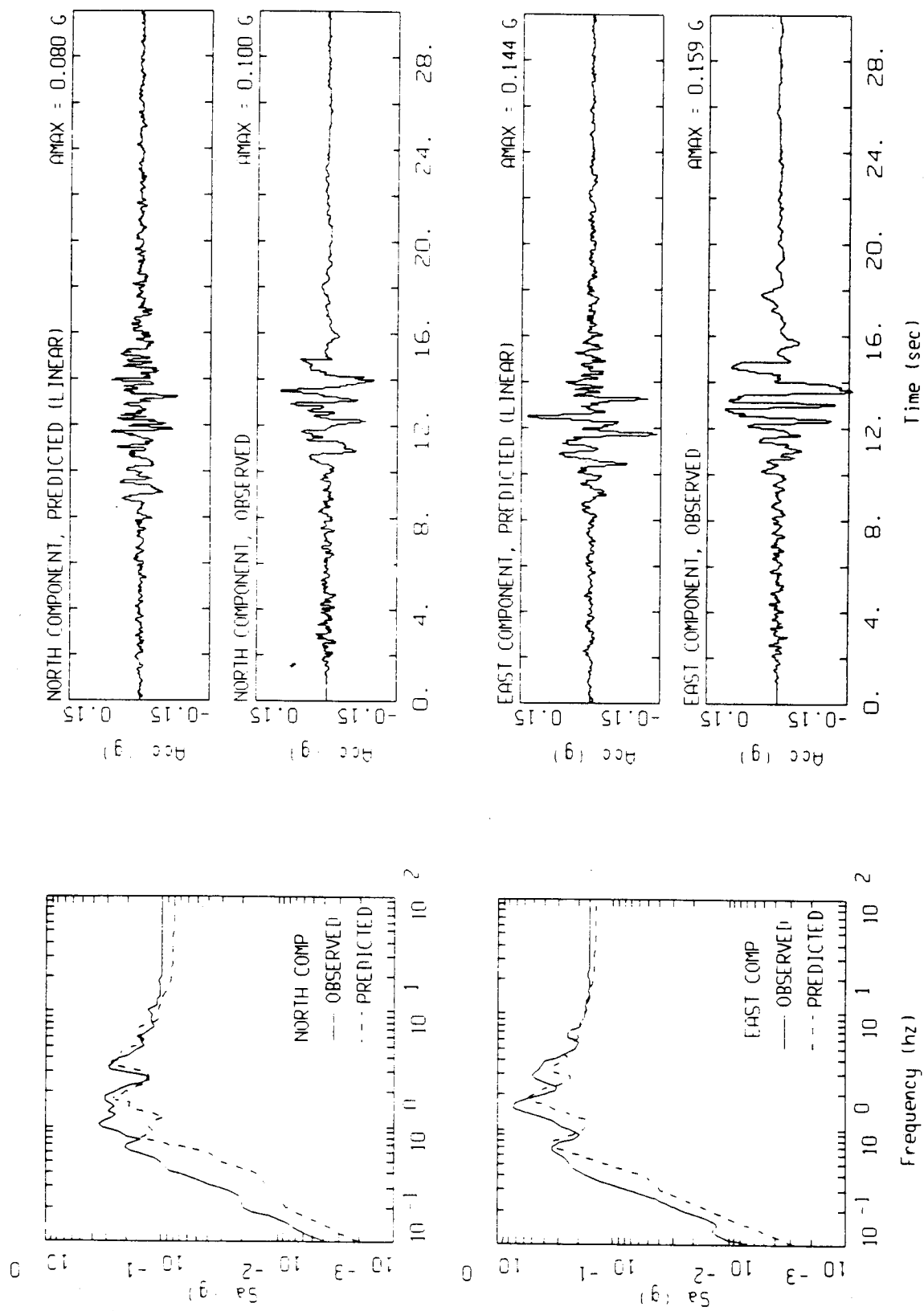


Figure 6.B-19 Comparison of observed and predicted time histories and 5% damped spectral accelerations for the Loma Prieta earthquake at reference site Treasure Island using Yerba Buena Island rock outcrop as control motions. RASCAL linear analysis.

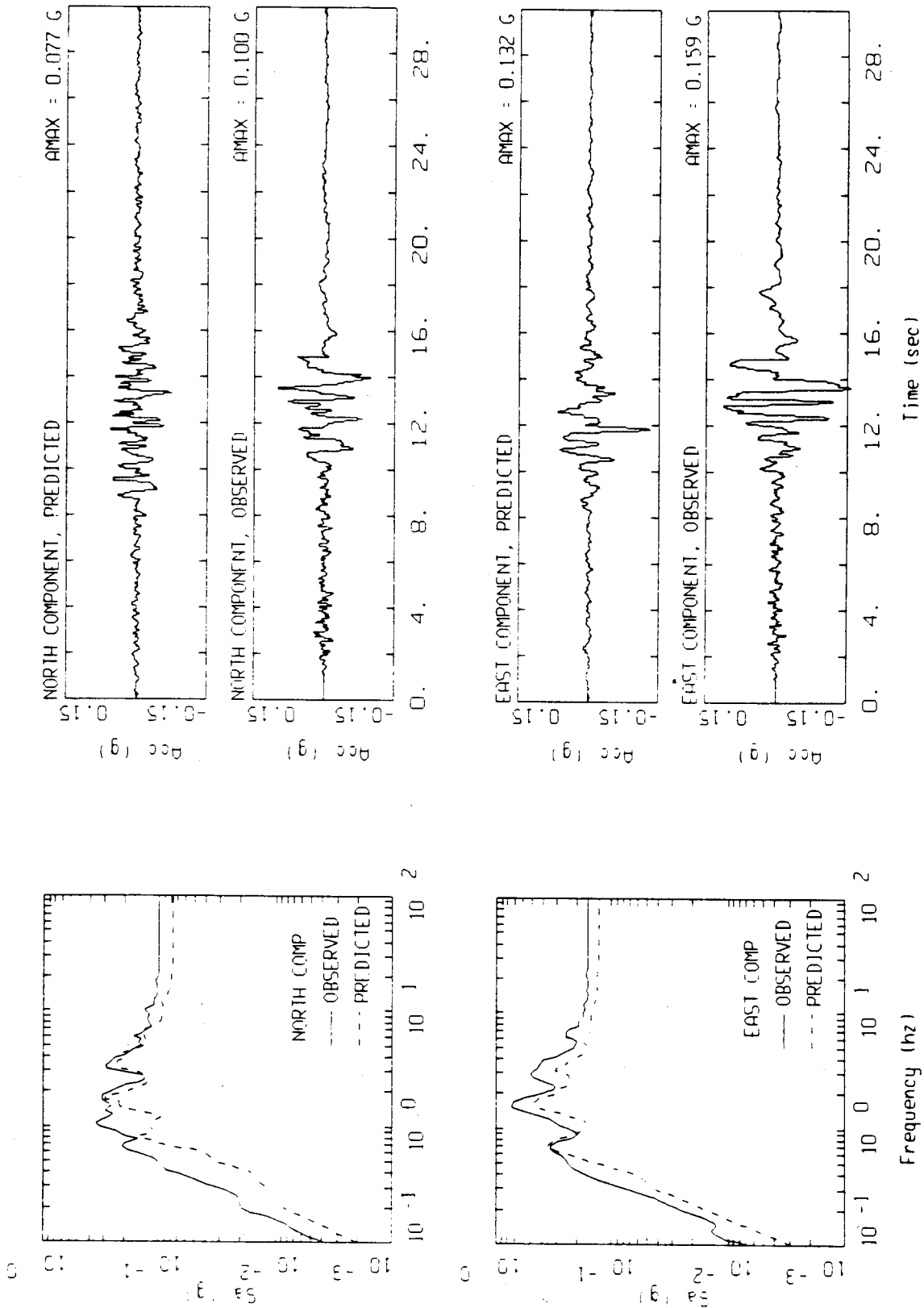


Figure 6.B-20
Comparison of observed and predicted time histories and 5% damped spectral accelerations for the Loma Prieta earthquake at reference site Treasure Island using Yerba Buena Island rock outcrop as control motions. RASCAL equivalent-linear analysis.

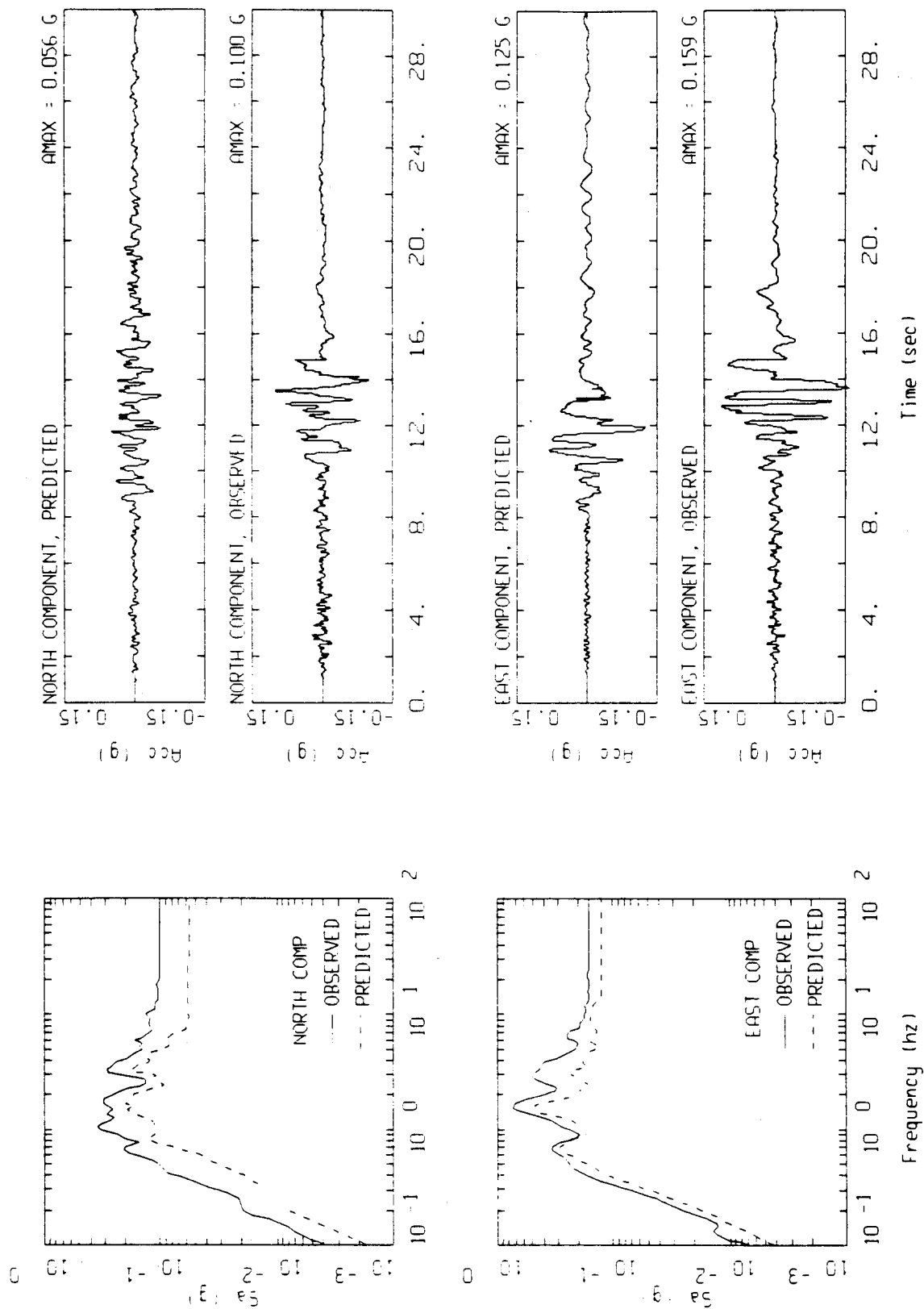


Figure 6.B-21
 Comparison of observed and predicted time histories and 5% damped spectral accelerations for the Loma Prieta earthquake at reference site Treasure Island using Yerba Buena Island rock outcrop as control motions. DESRA analysis.

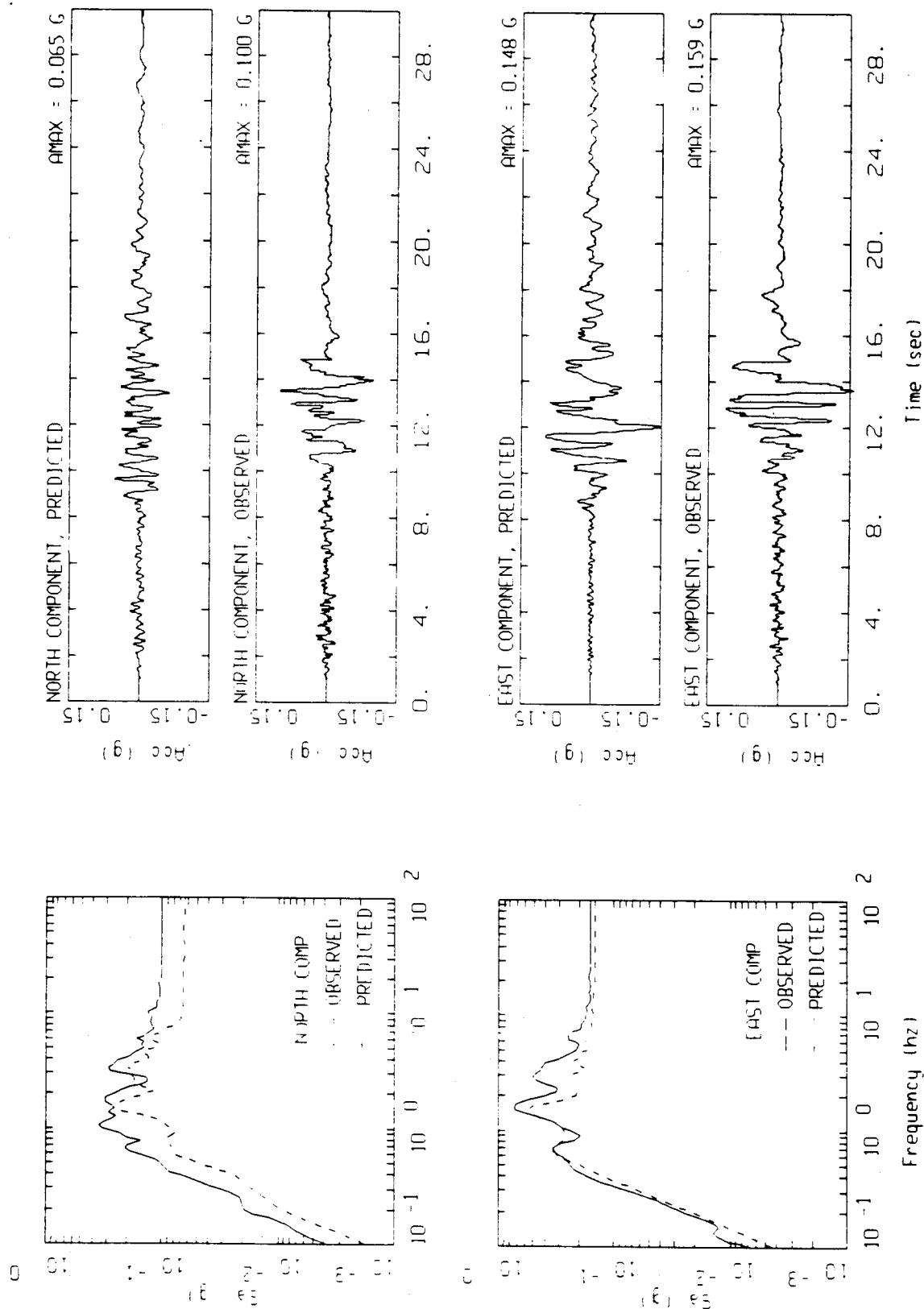


Figure 6.B-22

Comparison of observed and predicted time histories and 5% damped spectral accelerations for the Loma Prieta earthquake at reference site Treasure Island using Yerba Buena Island rock outcrop as control motions. SUMDES analysis.

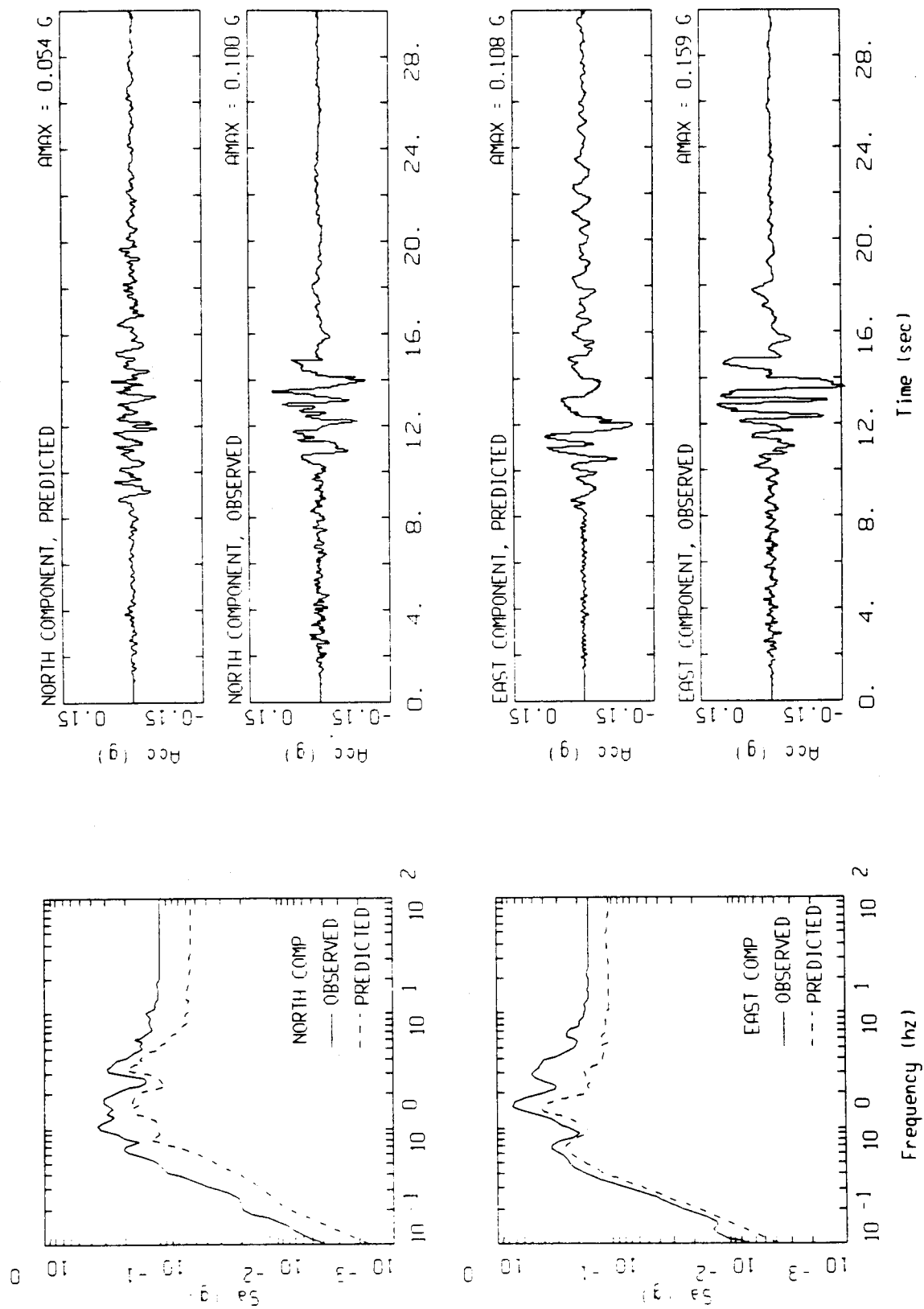


Figure 6.B-23

Comparison of observed and predicted time histories and 5% damped spectral accelerations for the Loma Prieta earthquake at reference site Treasure Island using Yerba Buena Island rock outcrop as control motions. TESS analysis.

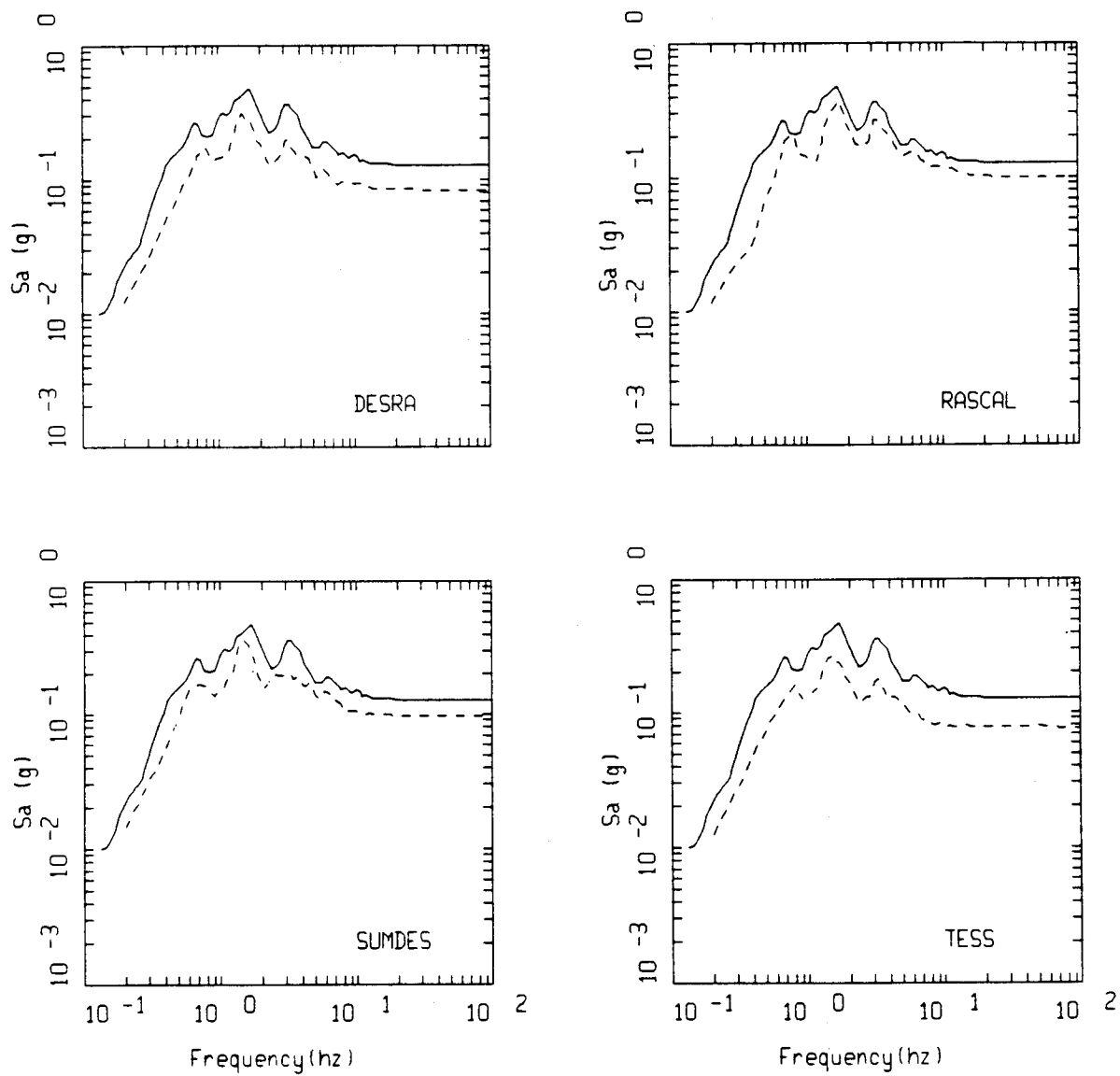


Figure 6.B-24

Comparison of log average (horizontal components) 5% damped spectral accelerations for the four analyses to the empirical spectra at reference site Treasure Island for the Loma Prieta earthquake (empirical, solid; predicted, dashed).

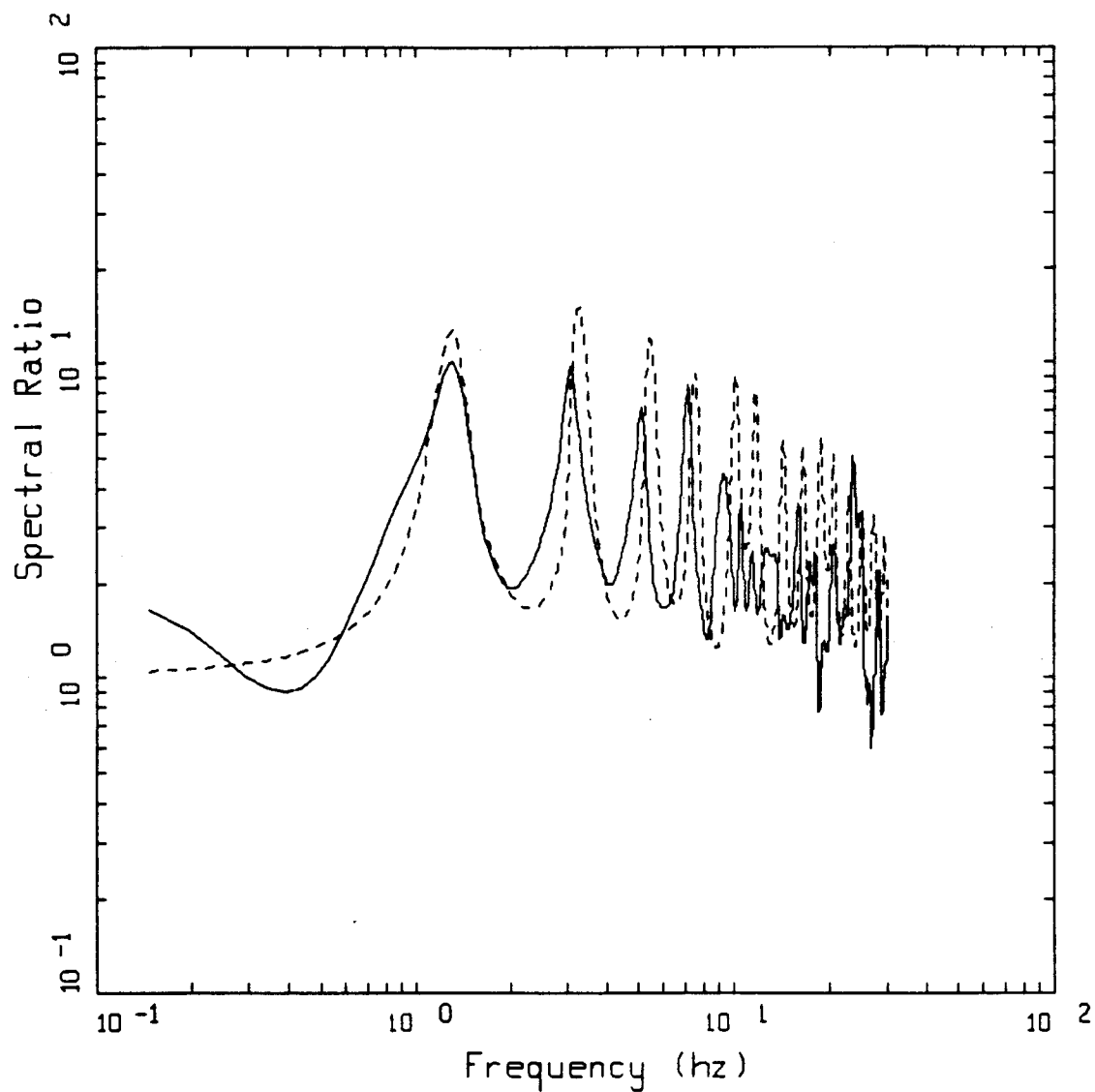


Figure 6.B-25

Fourier amplitude spectral ratios, surface-to-154 ft, at reference site Lotung from LSST event 10 (solid) and computed using base-case shear-wave velocity profile: linear analysis. Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

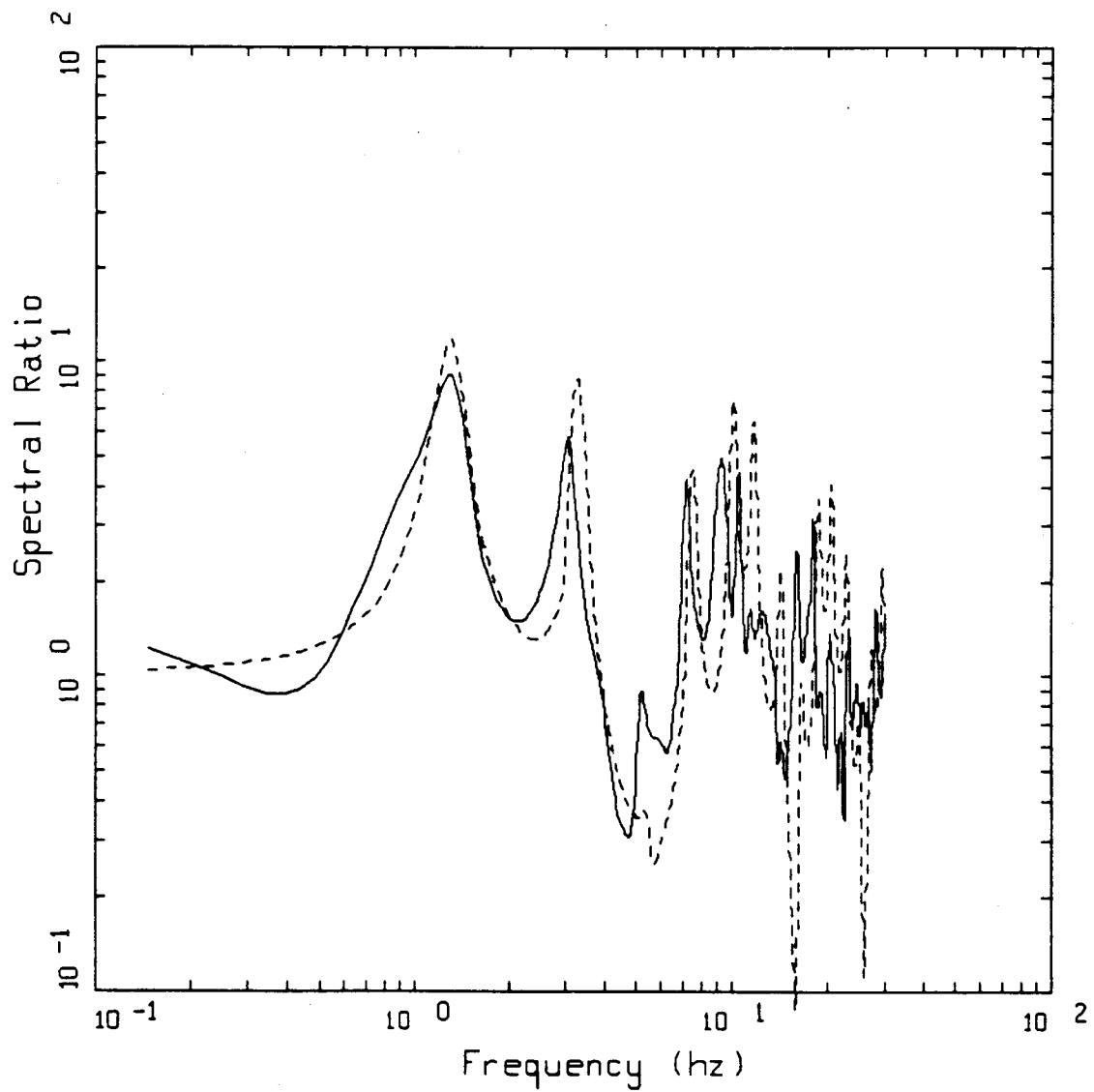


Figure 6.B-26

Fourier amplitude spectral ratios, 20-to-154 ft, at reference site Lotung from LSST event 10 (solid) and computed using base-case shear-wave velocity profile: linear analysis. Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

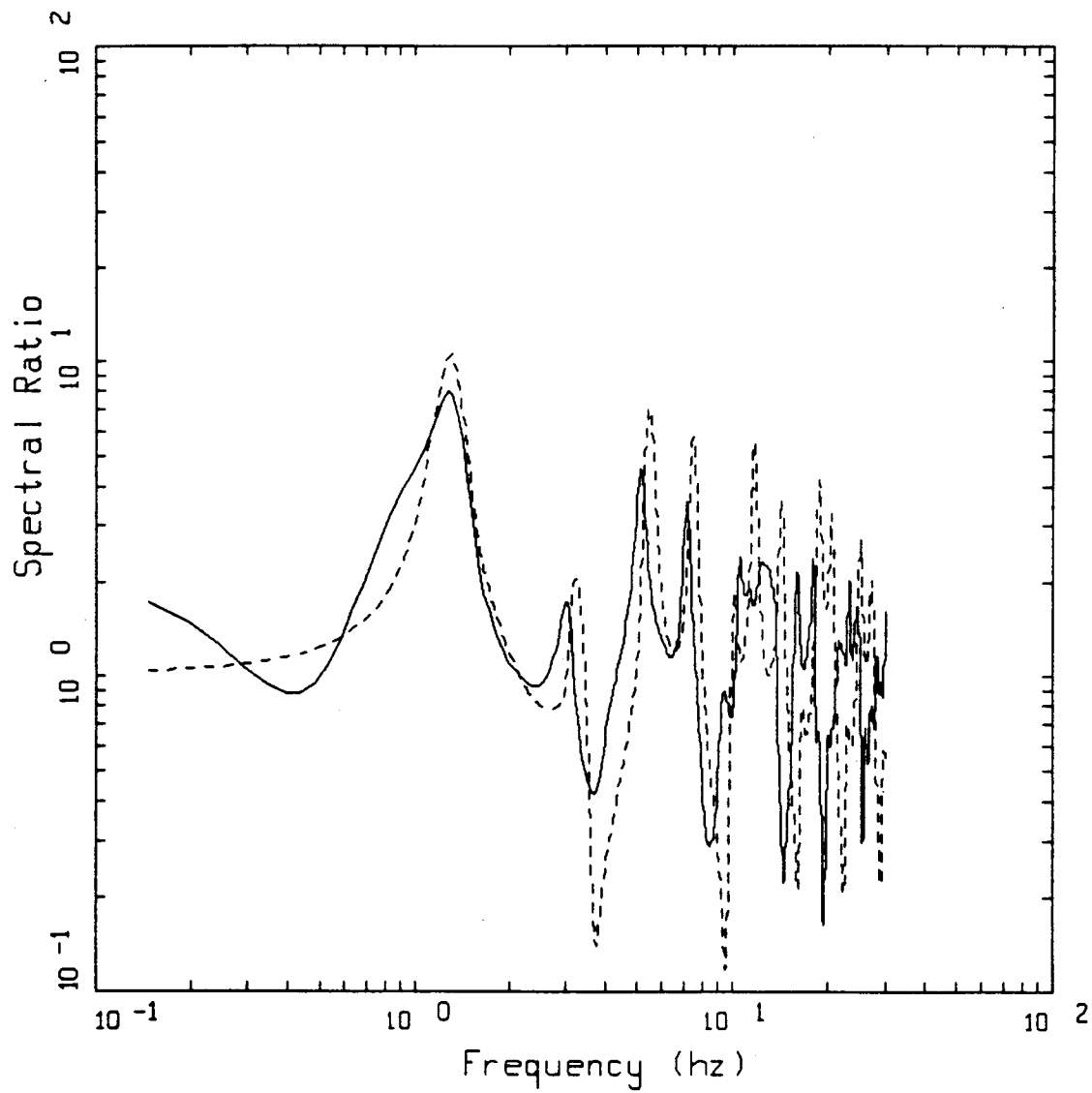


Figure 6.B-27

Fourier amplitude spectral ratios, 36-to-154 ft, at reference site Lotung from LSST event 10 (solid) and computed using base-case shear-wave velocity profile: linear analysis. Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

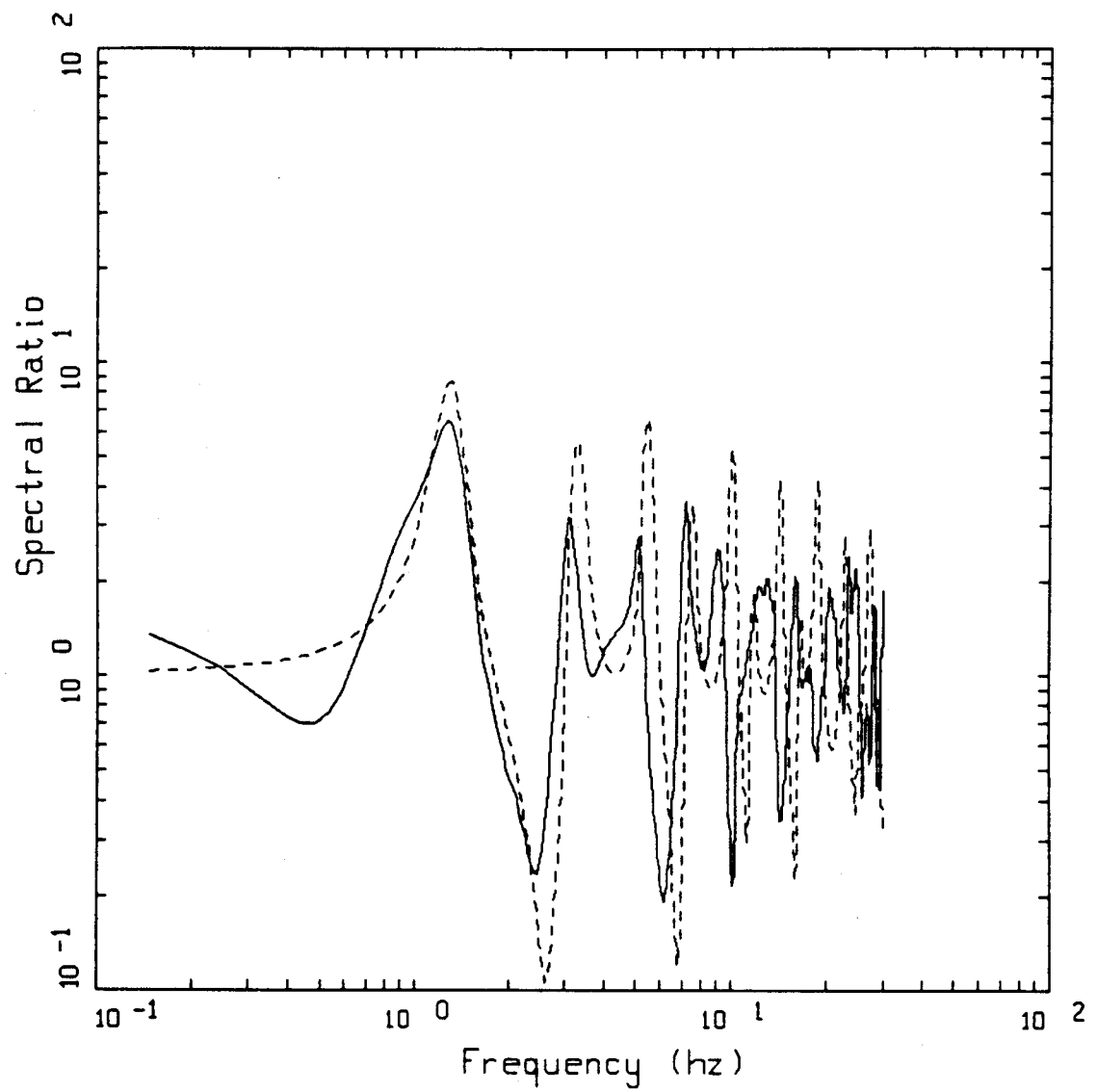


Figure 6.B-28

Fourier amplitude spectral ratios, 56-to-154 ft, at reference site Lotung from LSST event 10 (solid) and computed using base-case shear-wave velocity profile: linear analysis. Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

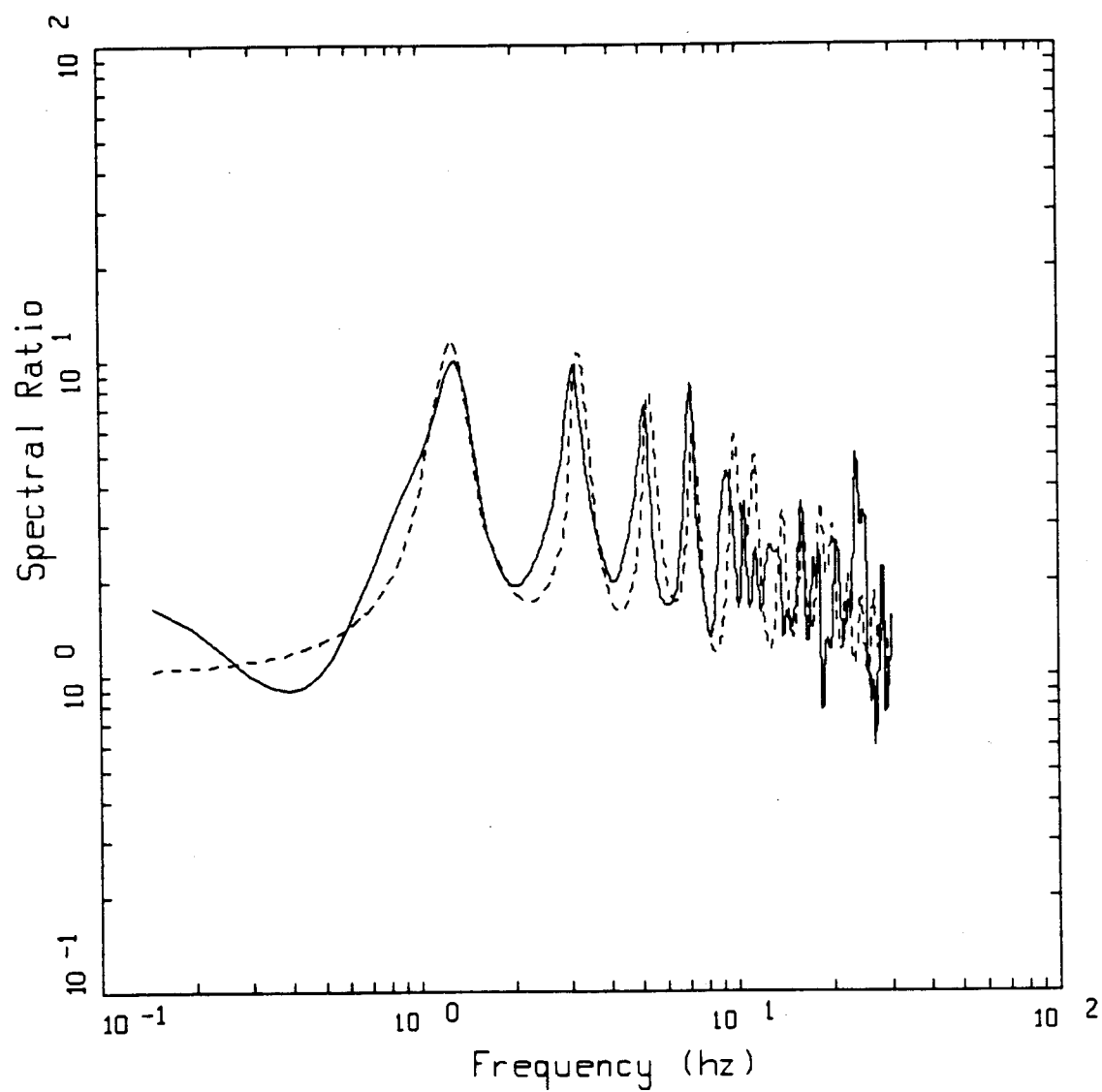


Figure 6.B-29

Fourier amplitude spectral ratios, surface-to-154 ft, at reference site Lotung from LSST event 10 (solid) and computed using base-case shear-wave velocity profile: equivalent-linear analysis. Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

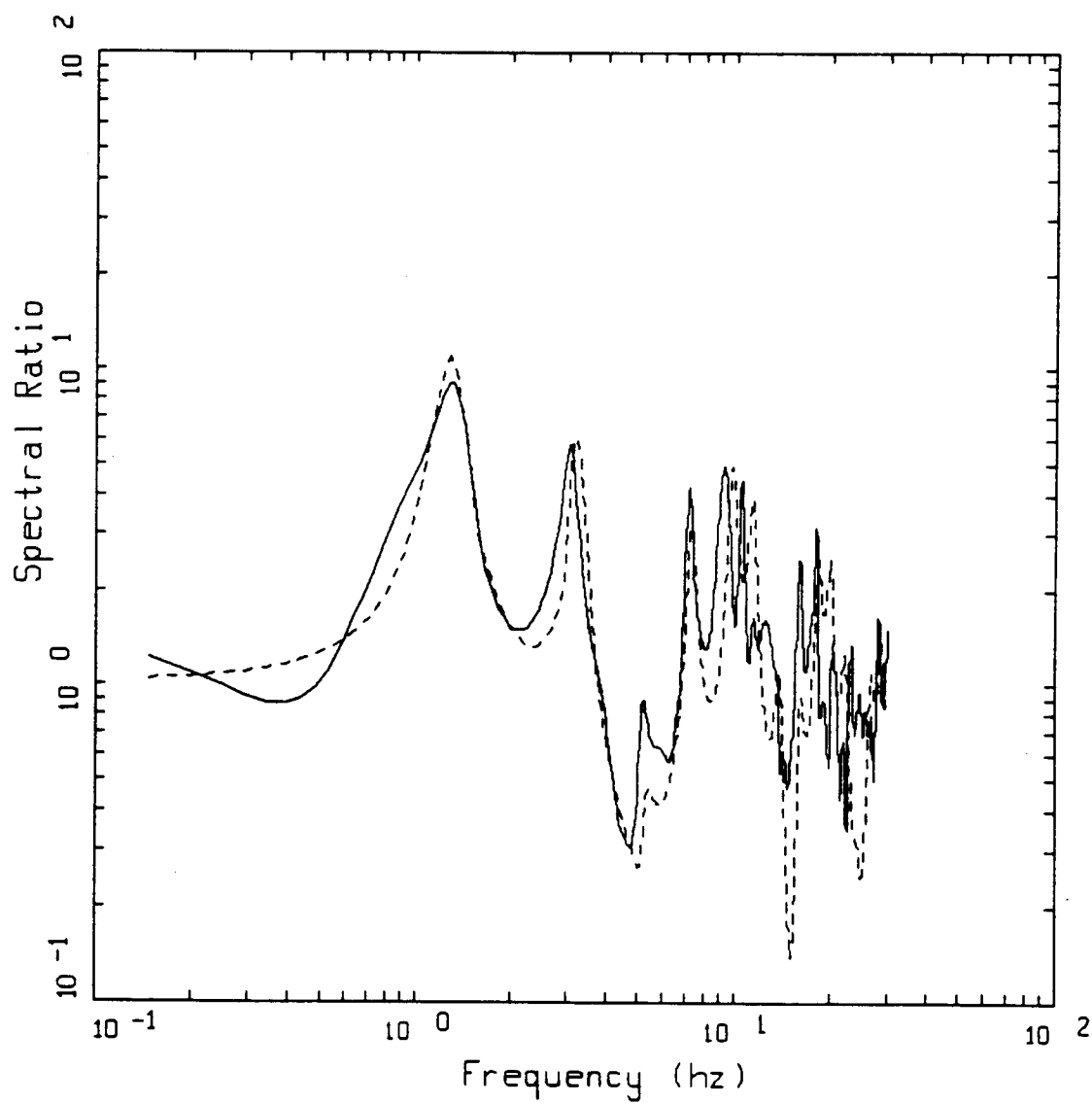


Figure 6.B-30

Fourier amplitude spectral ratios, 20-to-154 ft, at reference site Lotung from LSST event 10 (solid) and computed using base-case shear-wave velocity profile: equivalent-linear analysis. Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

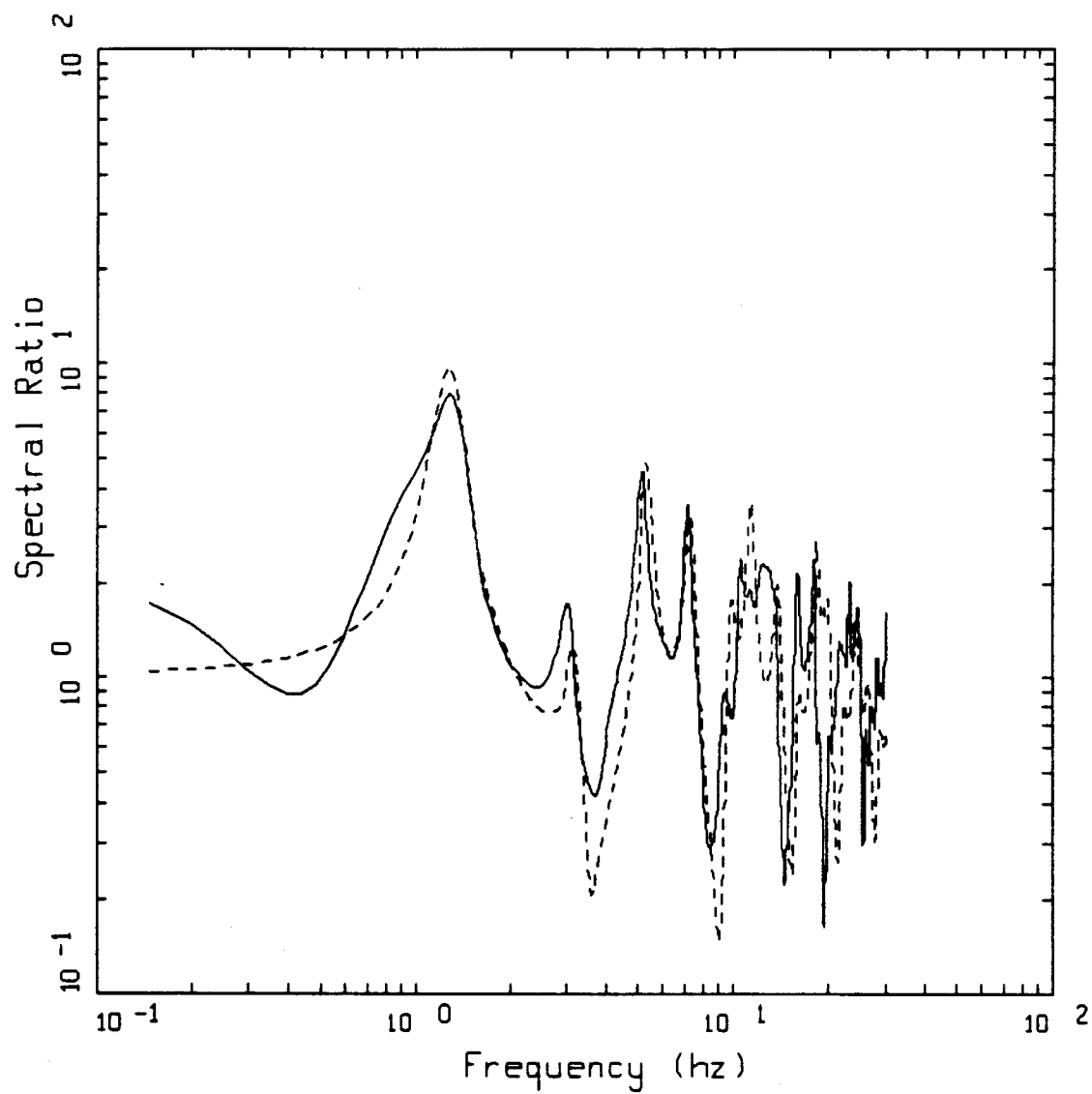


Figure 6.B-31

Fourier amplitude spectral ratios, 36-to-154 ft, at reference site Lotung from LSST event 10 (solid) and computed using base-case shear-wave velocity profile: equivalent-linear analysis. Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

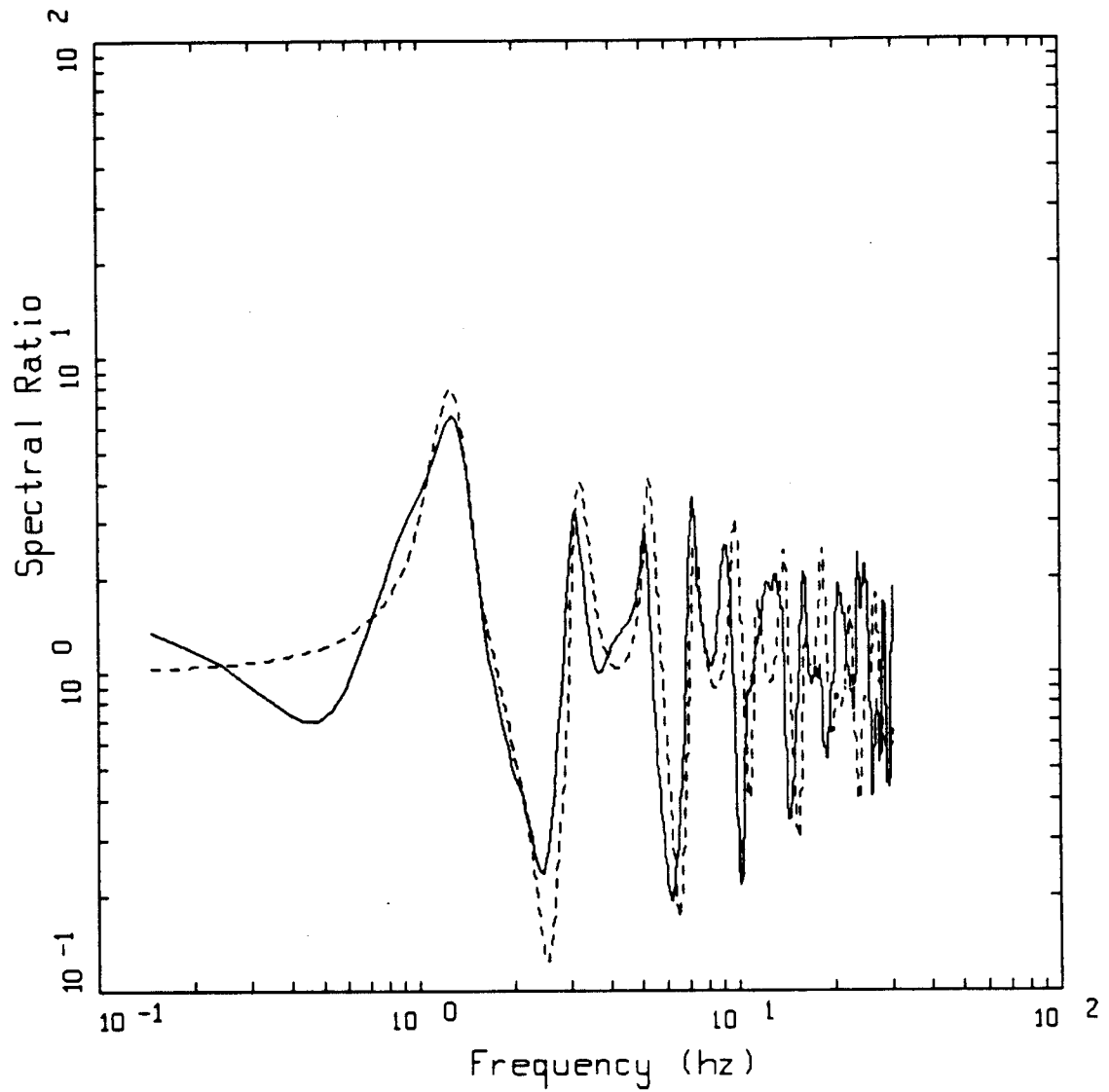


Figure 6.B-32

Fourier amplitude spectral ratios, 56-to-154 ft, at reference site Lotung from LSST event 10 (solid) and computed using base-case shear-wave velocity profile: equivalent-linear analysis. Numerator and denominator are smoothed with a 1/2 Hz wide triangular operator prior to taking ratios.

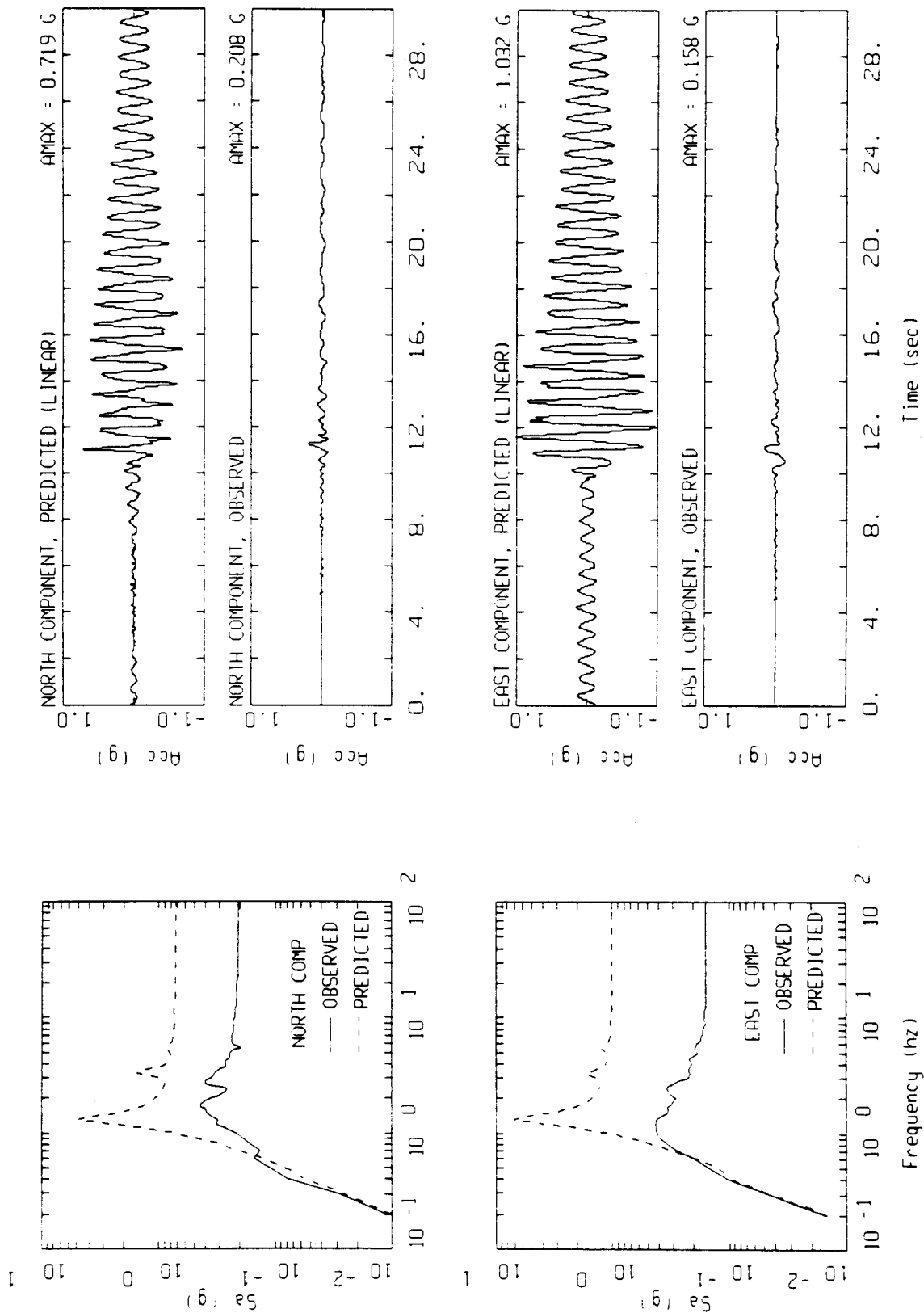


Figure 6.B-33

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for surface motions using the recordings at 154 ft as control motions. RASCAL linear analysis.

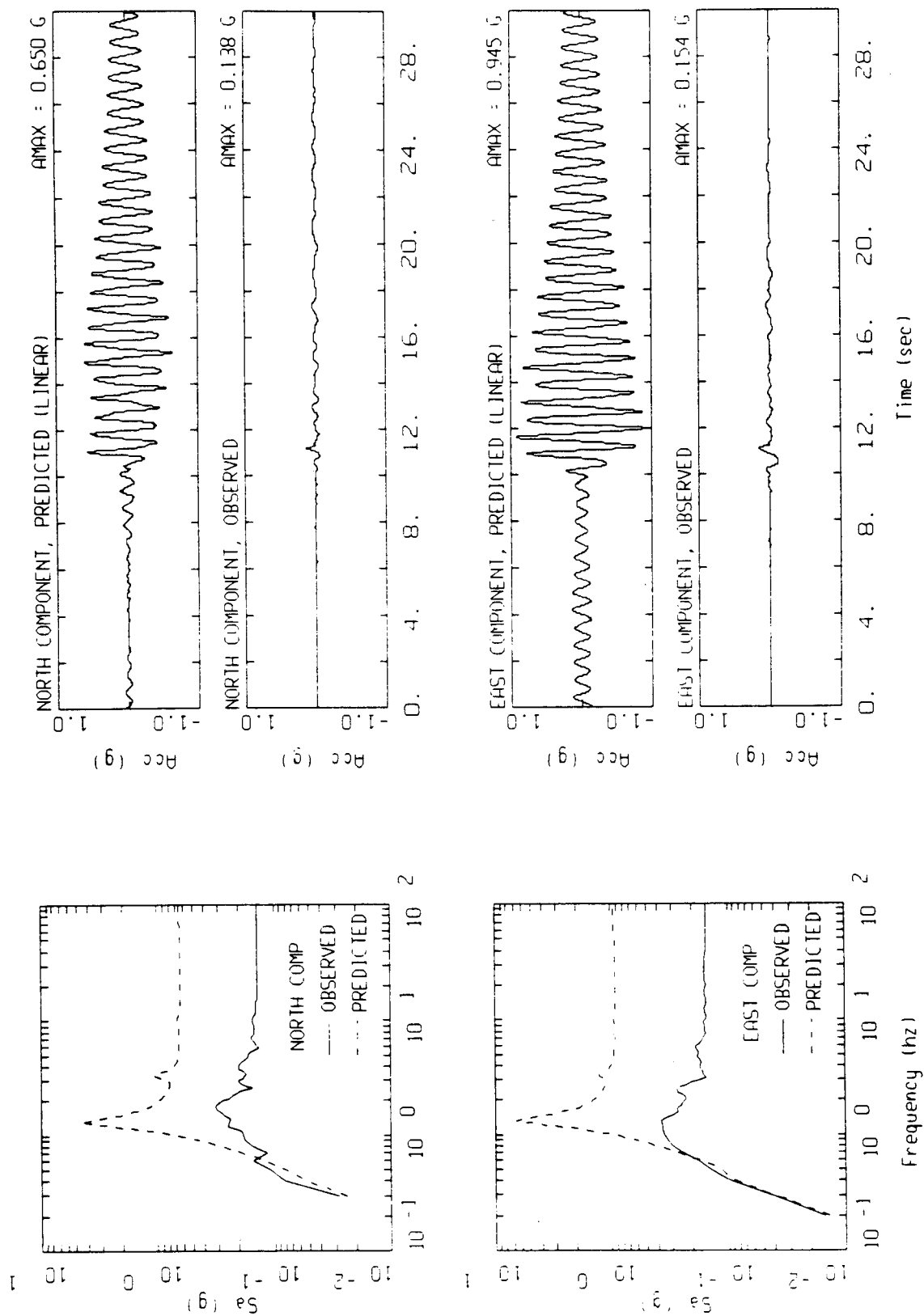


Figure 6.B-34

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 20 ft depth using the recordings at 154 ft as control motions. RASCAL linear analysis.

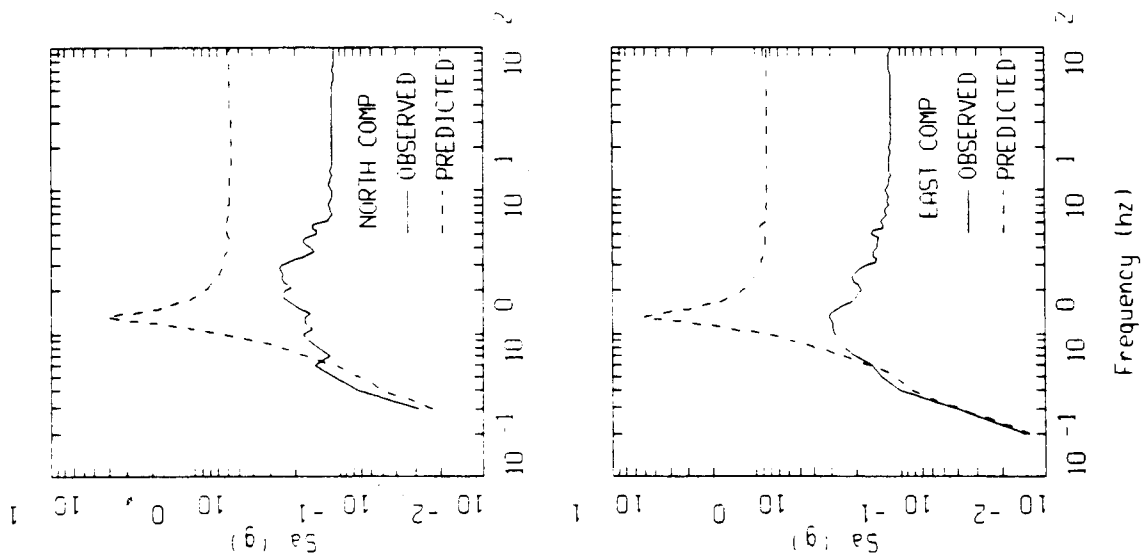


Figure 6.B-35

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 36 ft depth using the recordings at 154 ft as control motions. RASCAL linear analysis.

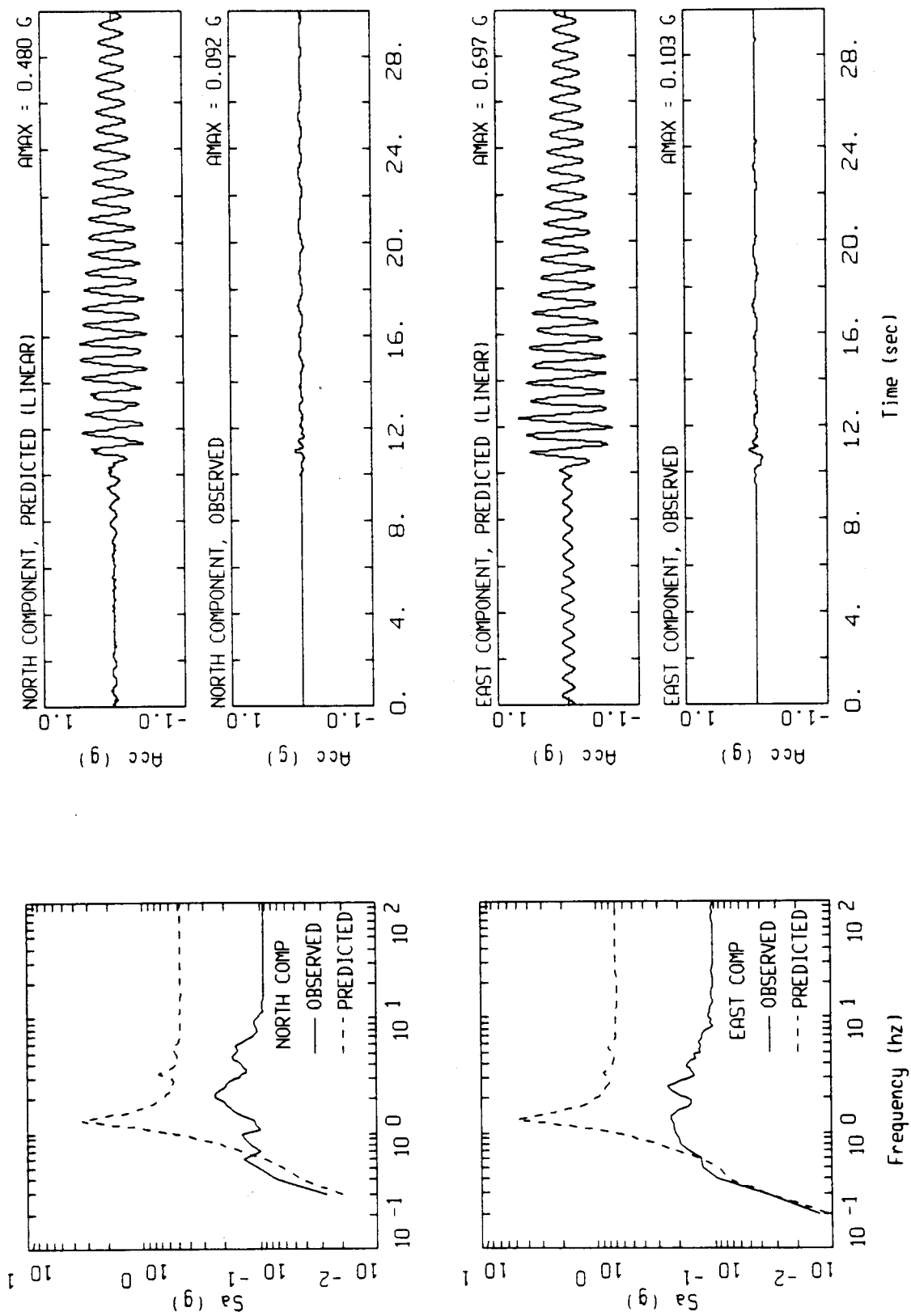


Figure 6.B-36
Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 56 ft depth using the recordings at 154 ft as control motions. RASCAL linear analysis.

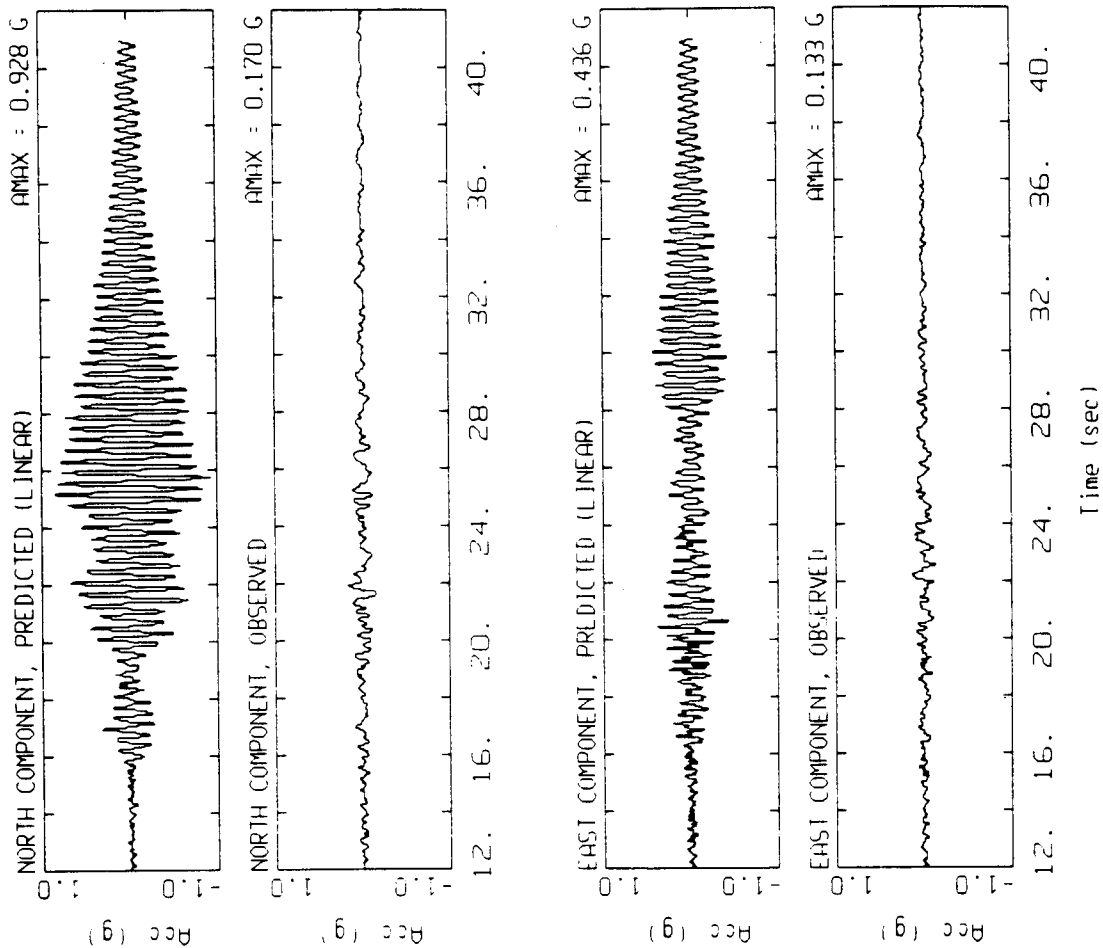
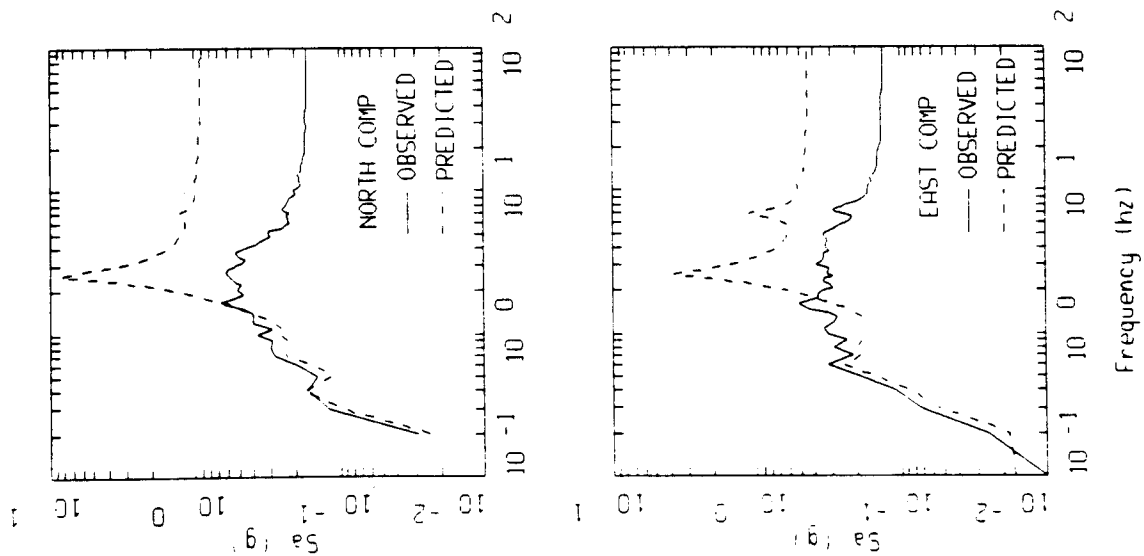


Figure 6.B-37

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for surface motions using the recordings at 154 ft as control motions. RASCAL linear analysis.

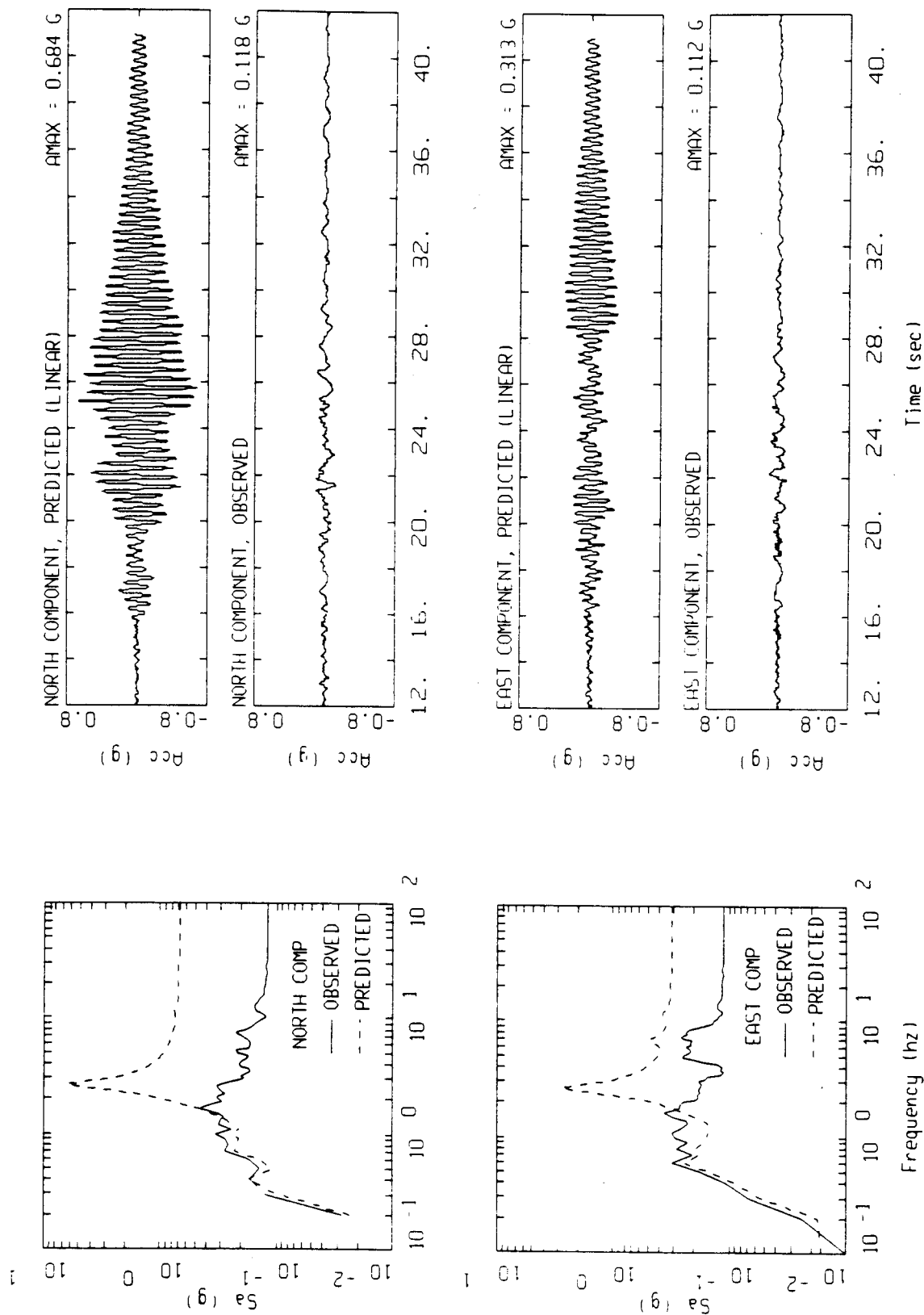


Figure 6.B-38

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for 20 ft depth using the recordings at 154 ft as control motions. RASCAL linear analysis.

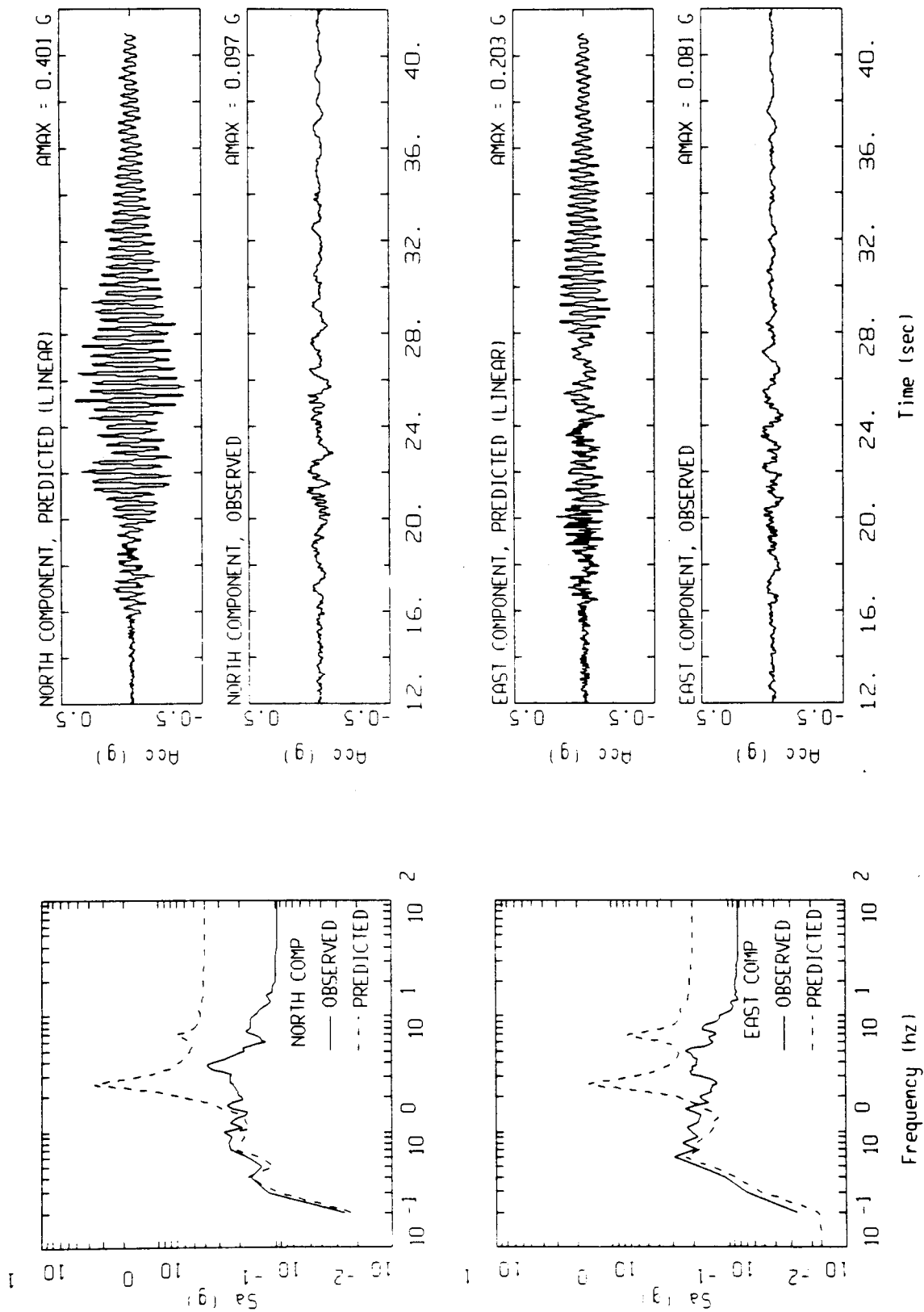


Figure 6.B-39
Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for 36 ft depth using the recordings at 154 ft as control motions. RASCAL linear analysis.

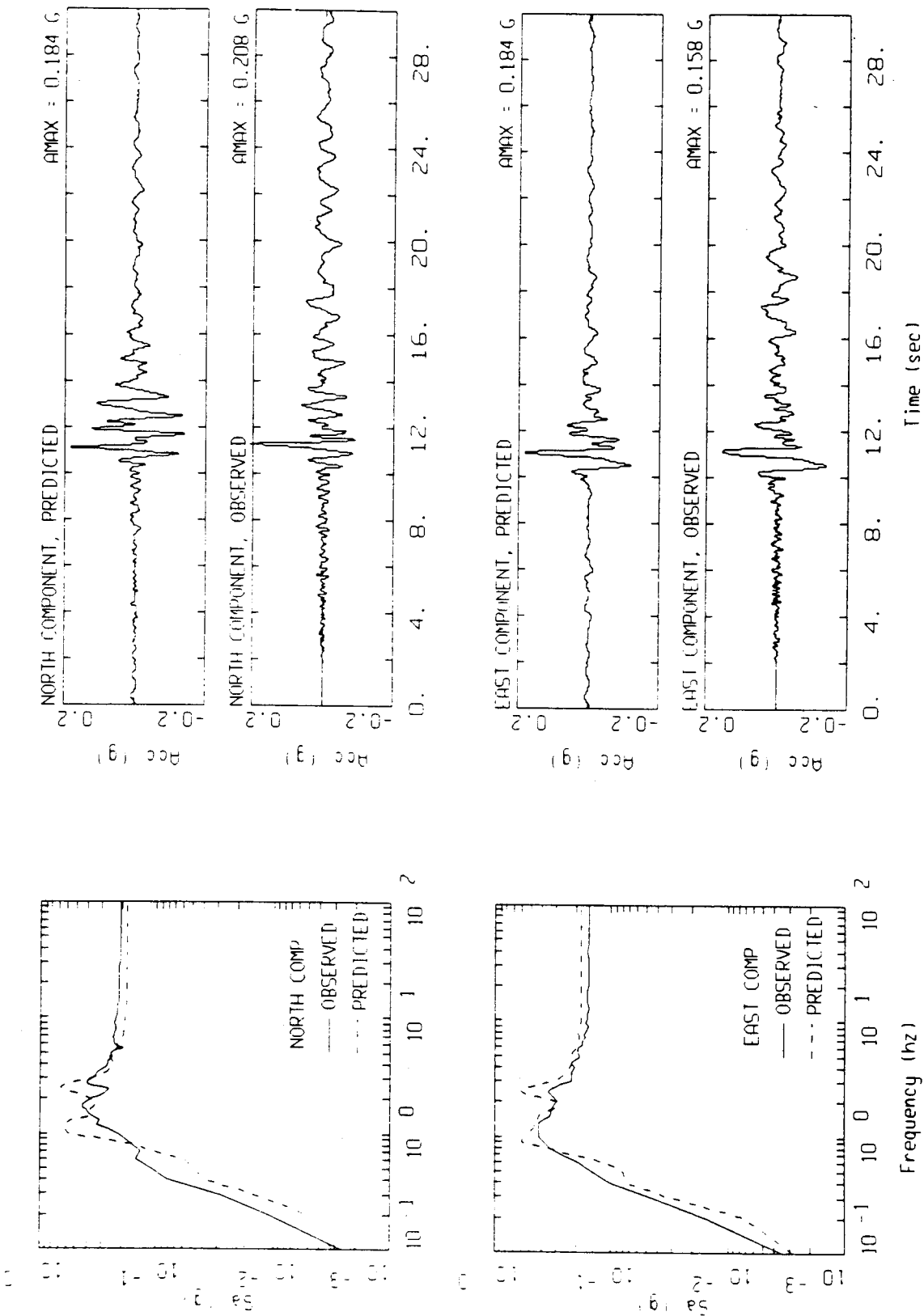


Figure 6.B-40

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSS event 7 at reference site Lotung for surface motions using the recordings at 154 ft as control motions. RASCAL equivalent-linear analysis.

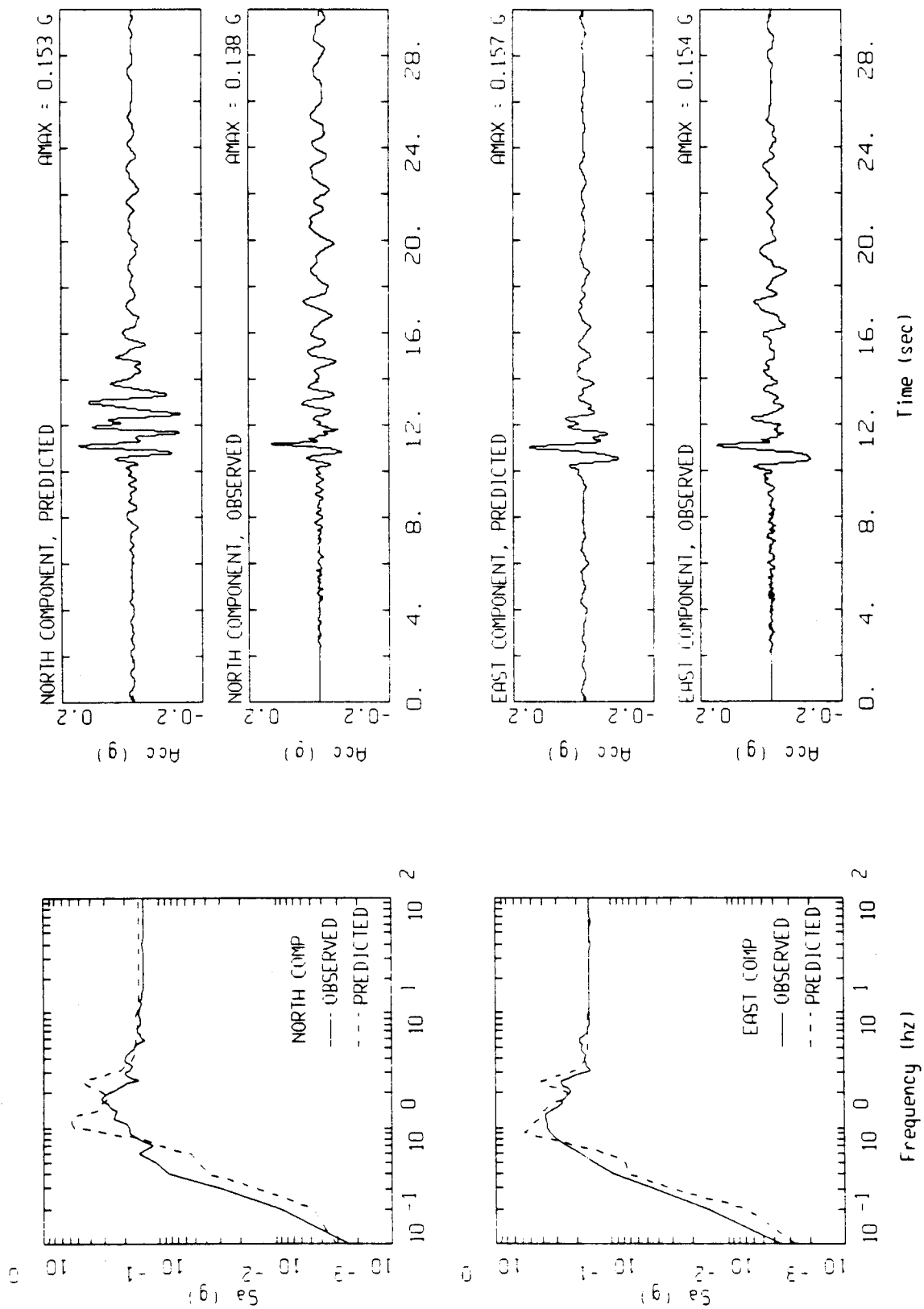


Figure 6.B-41
Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 20 ft depth using the recordings at 154 ft as control motions. RASCAL equivalent-linear analysis.

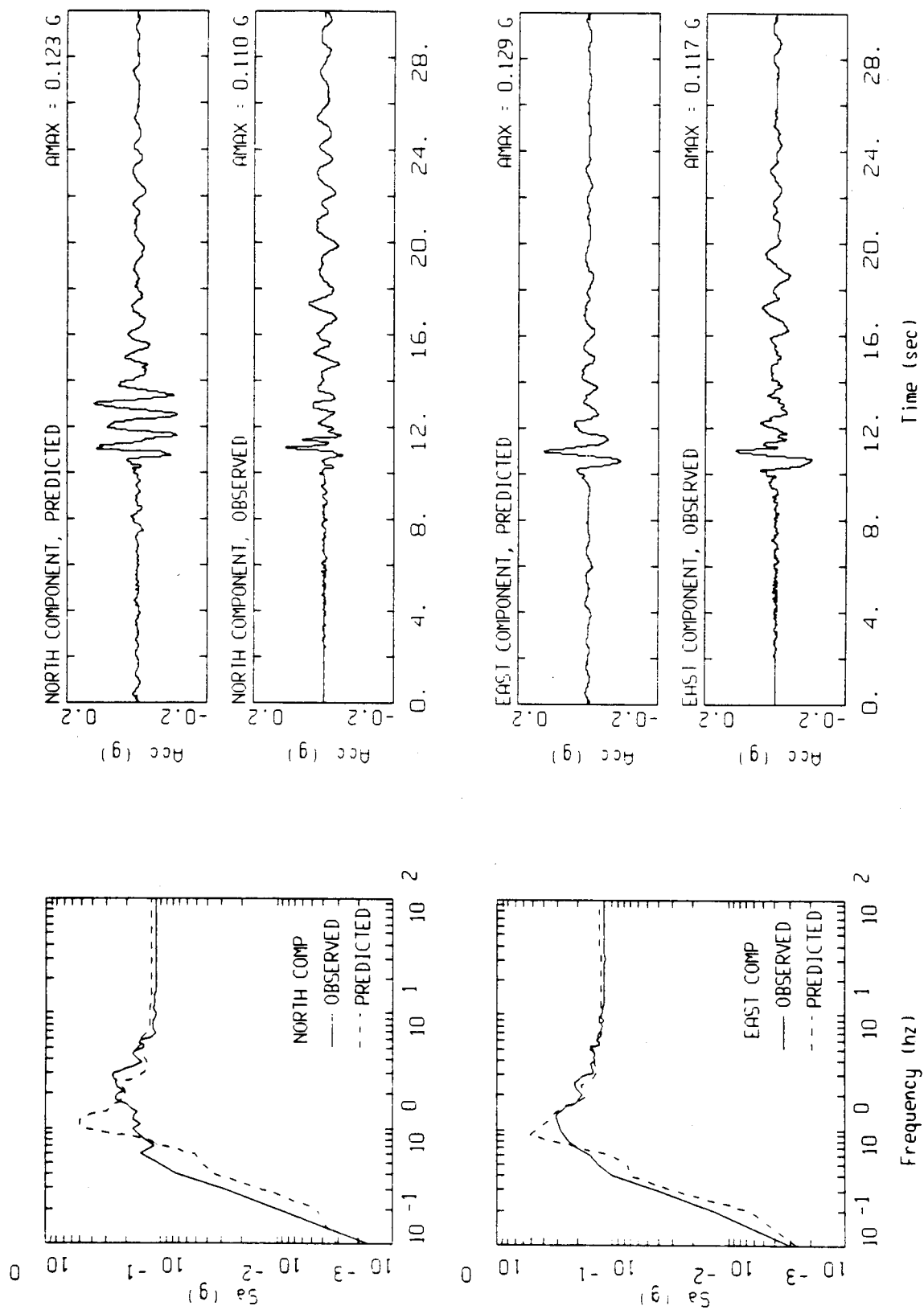


Figure 6.B-42

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 36 ft depth using the recordings at 154 ft as control motions. RASCAL equivalent-linear analysis.

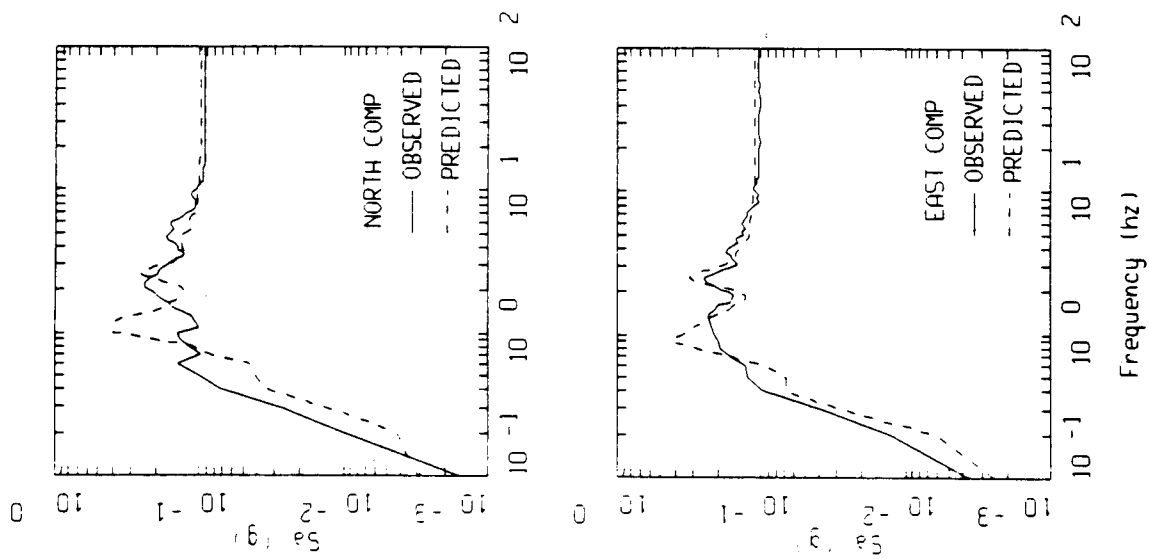


Figure 6.B-43

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 56 ft depth using the recordings at 154 ft as control motions. RASCAL equivalent-linear analysis.

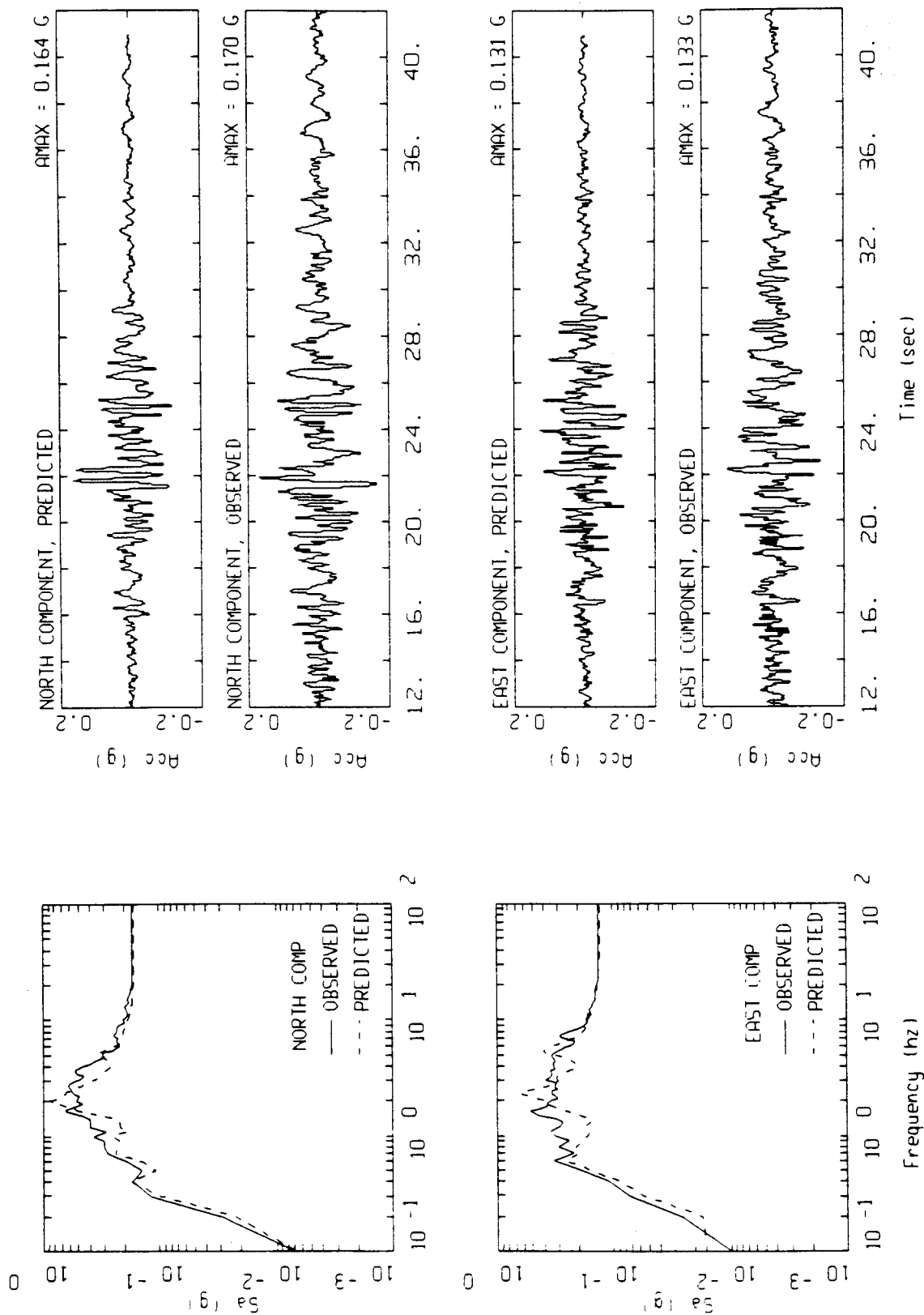


Figure 6.B-44

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for surface motions using the recordings at 154 ft as control motions. RASCAL equivalent-linear analysis.

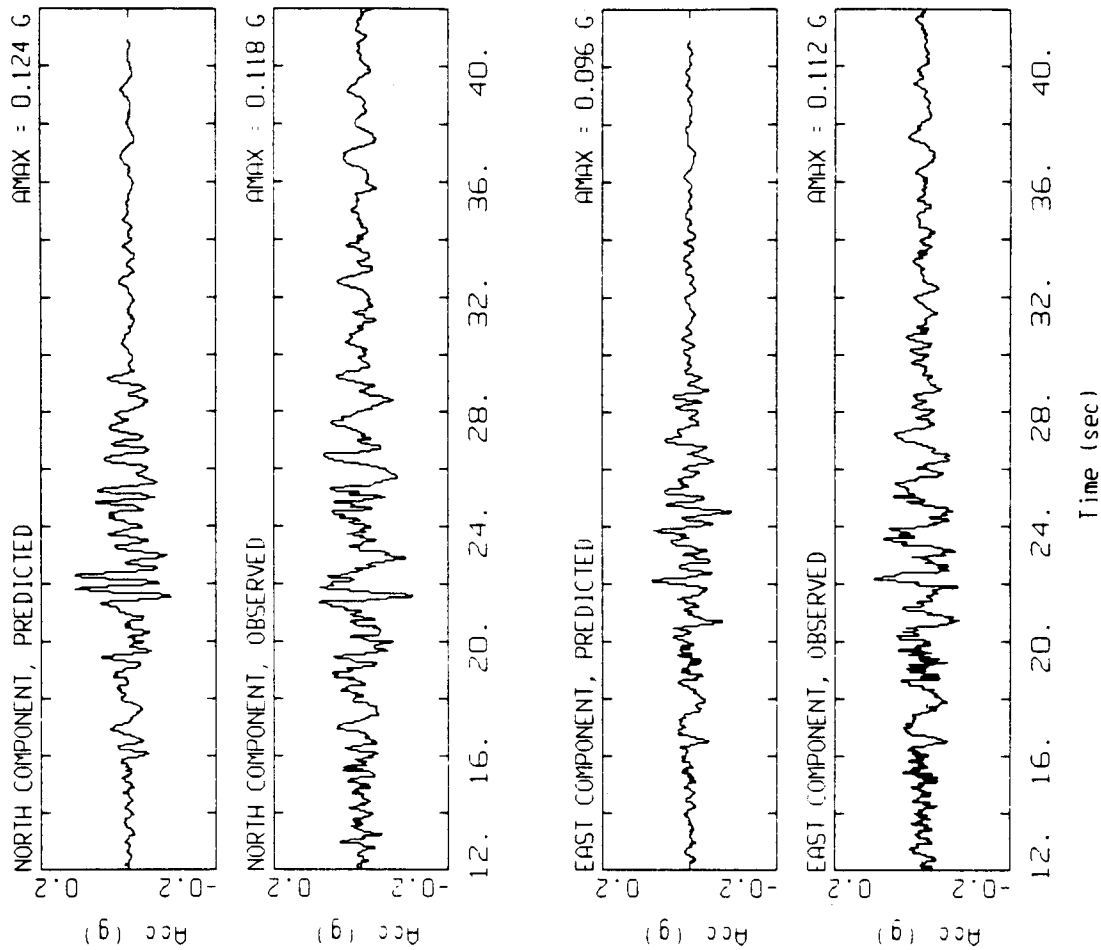
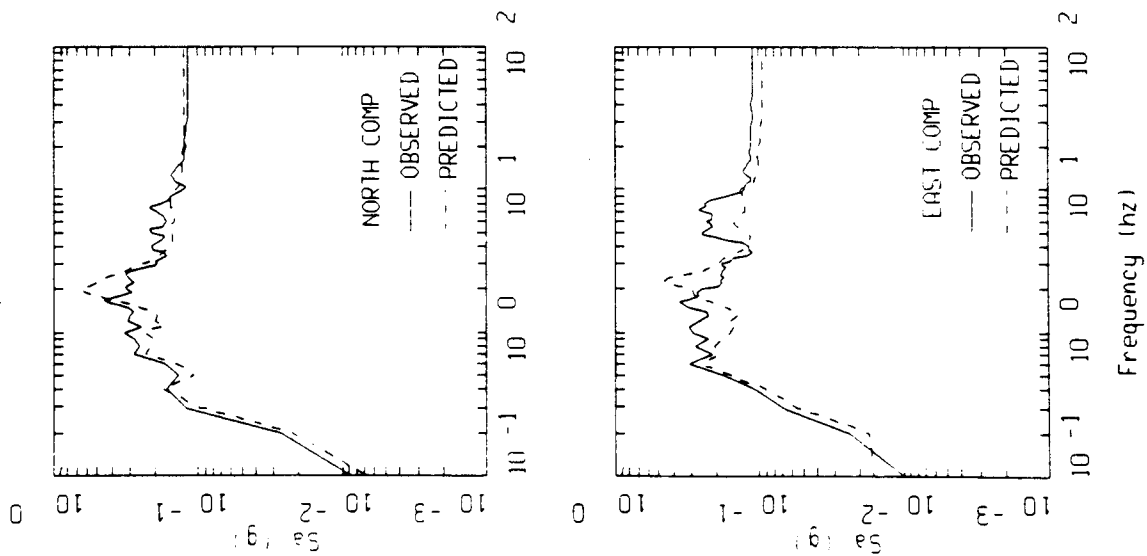


Figure 6.B-45

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 20 ft depth using the recordings at 154 ft as control motions. RASCAL equivalent-linear analysis.

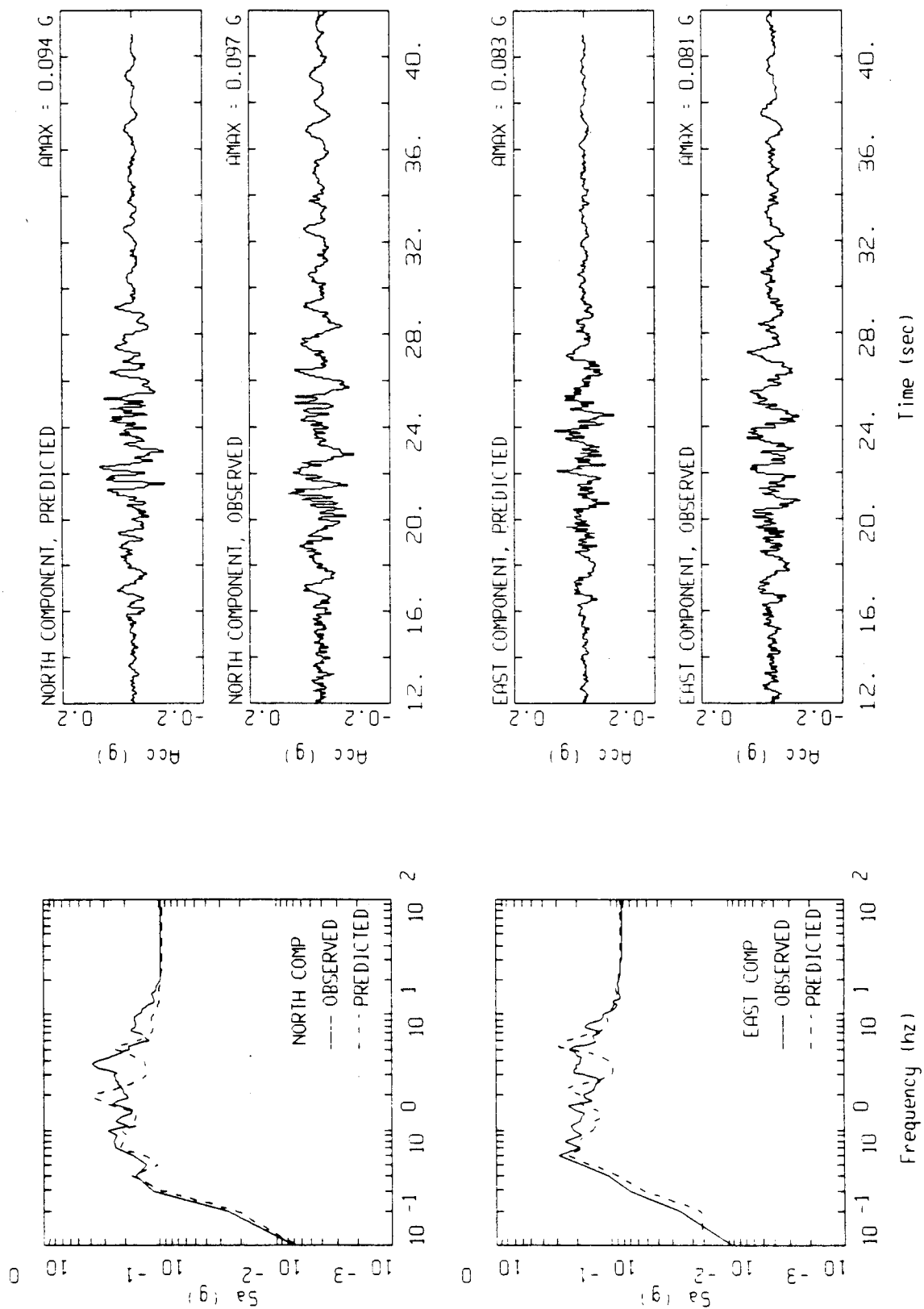


Figure 6.B-46

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for 36 ft depth using the recordings at 154 ft as control motions. RASCAL equivalent-linear analysis.

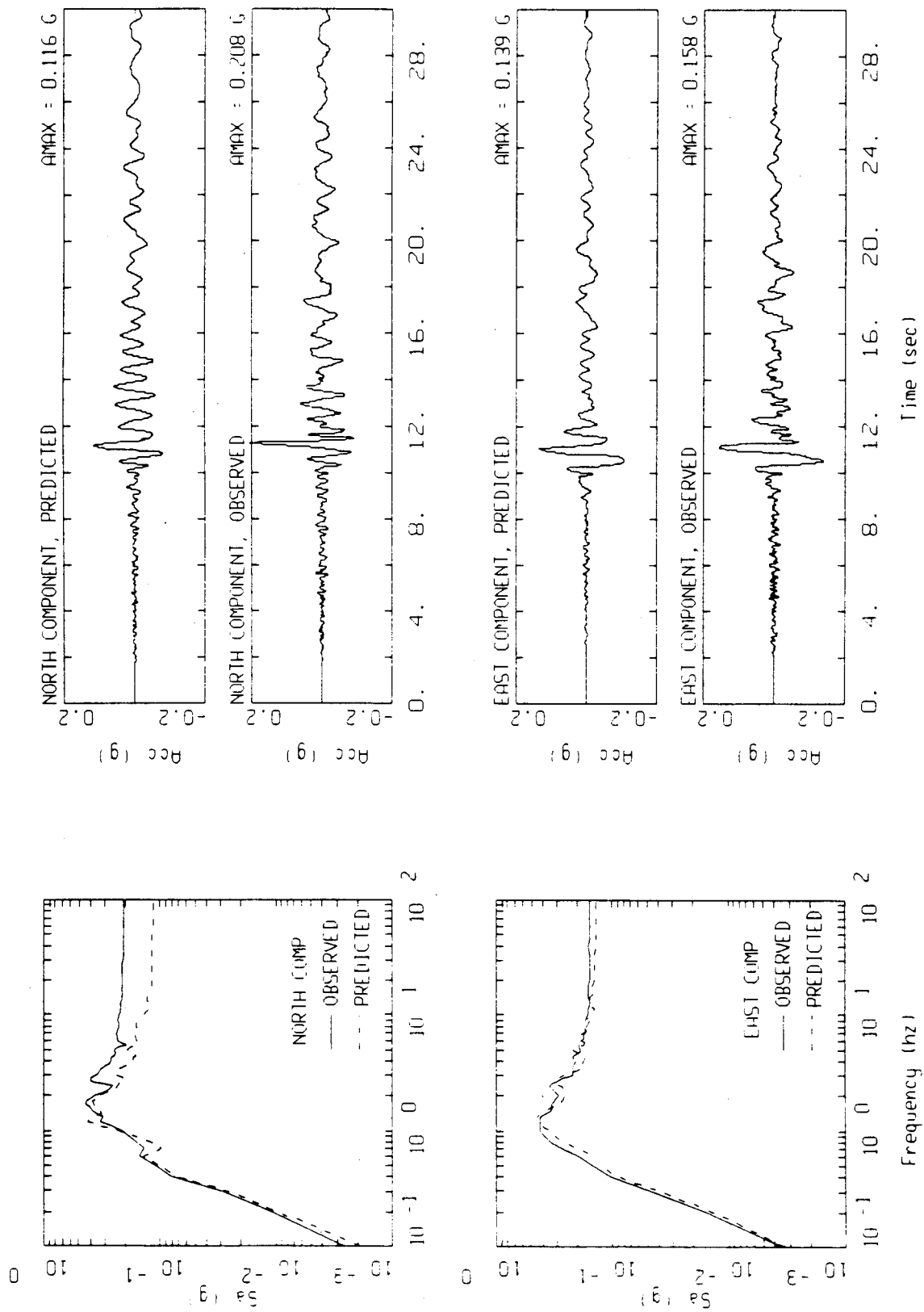


Figure 6.B-47

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for surface motions using the recordings at 154 ft as control motions. DESRA analysis.

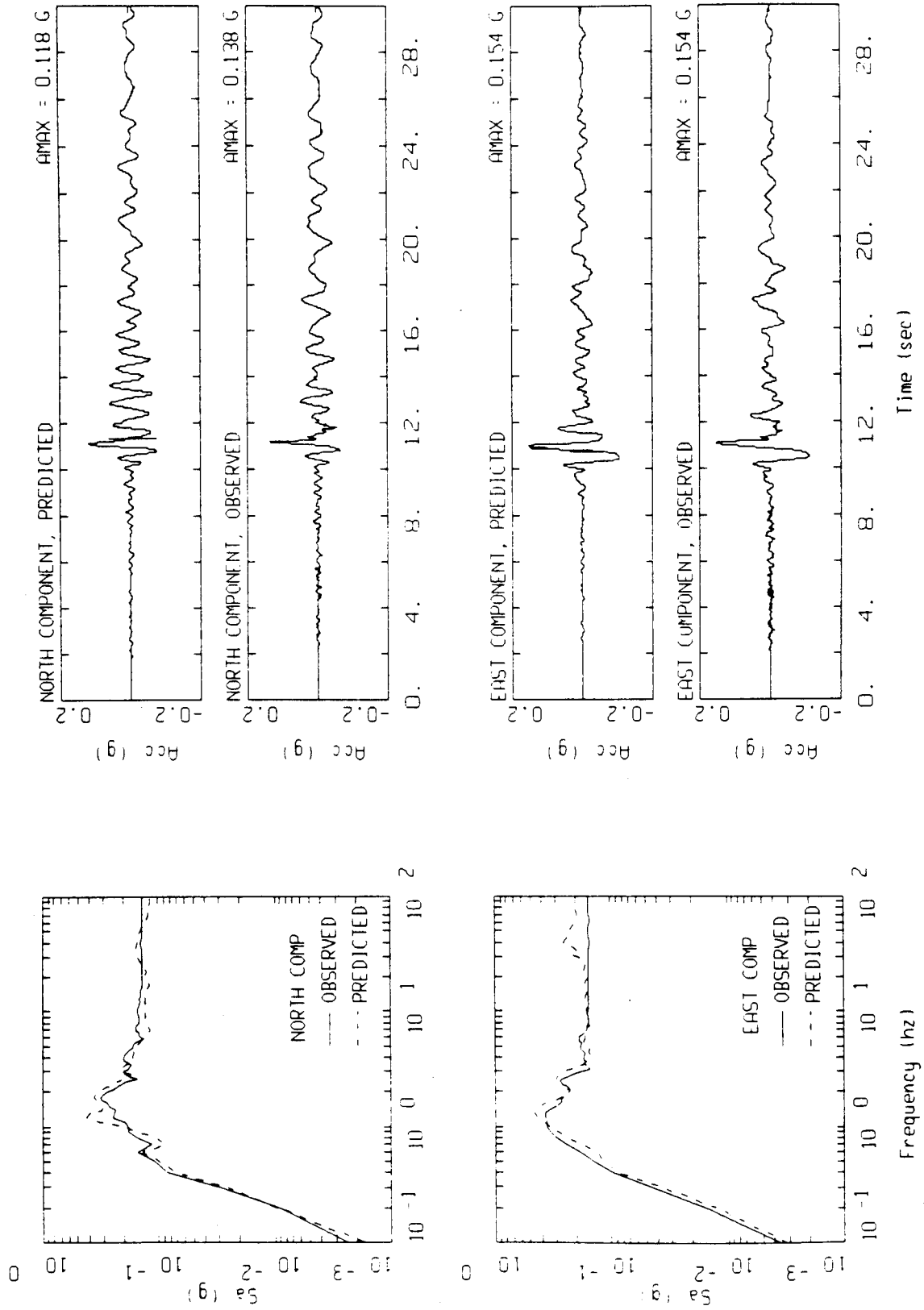


Figure 6.B-48
 Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 20 ft depth using the recordings at 154 ft as control motions. DESRA analysis.

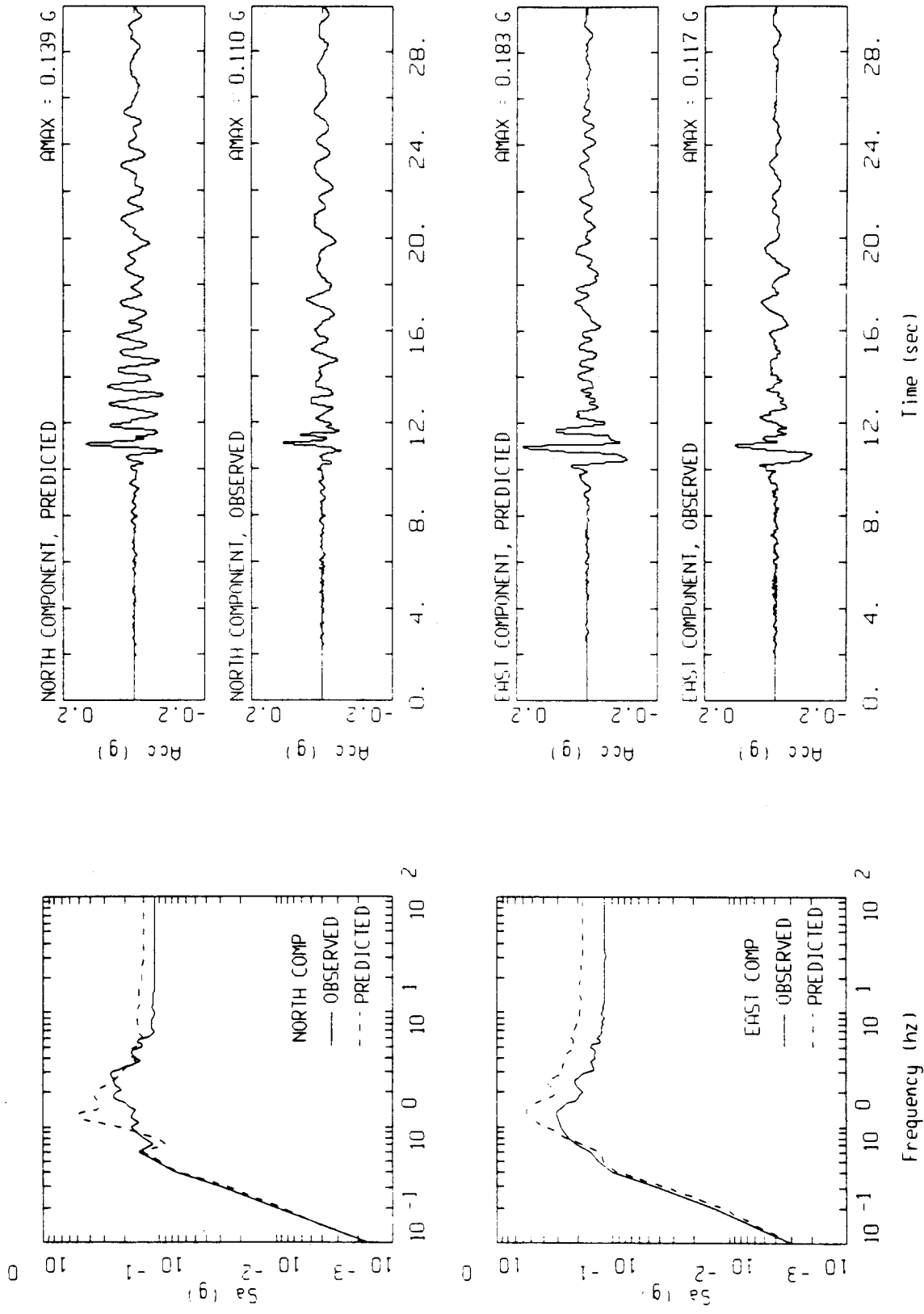


Figure 6.B-49
Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 36 ft depth using the recordings at 154 ft as control motions. DESRA analysis.

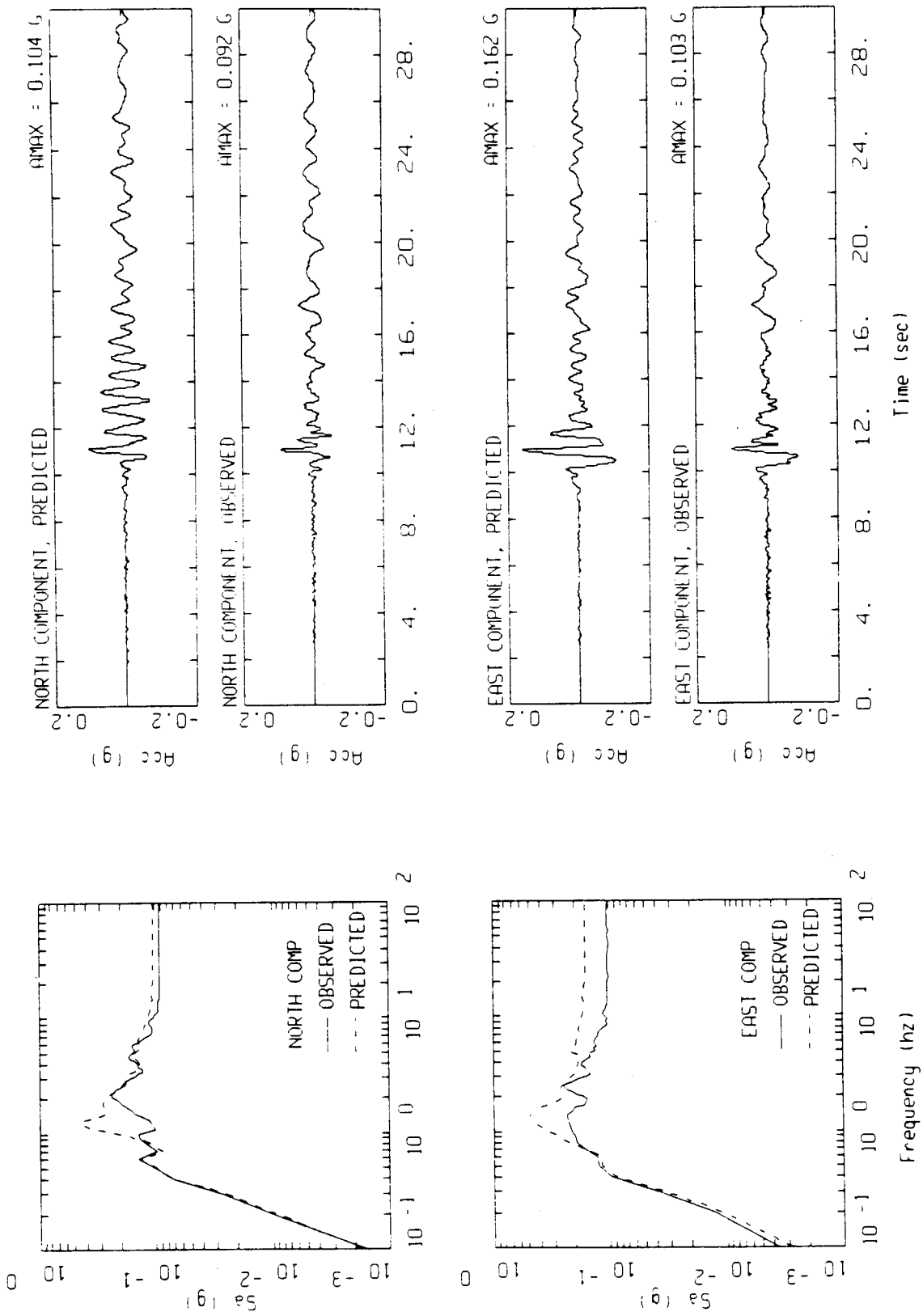


Figure 6.B-50
 Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 56 ft depth using the recordings at 154 ft as control motions. DESRA analysis.

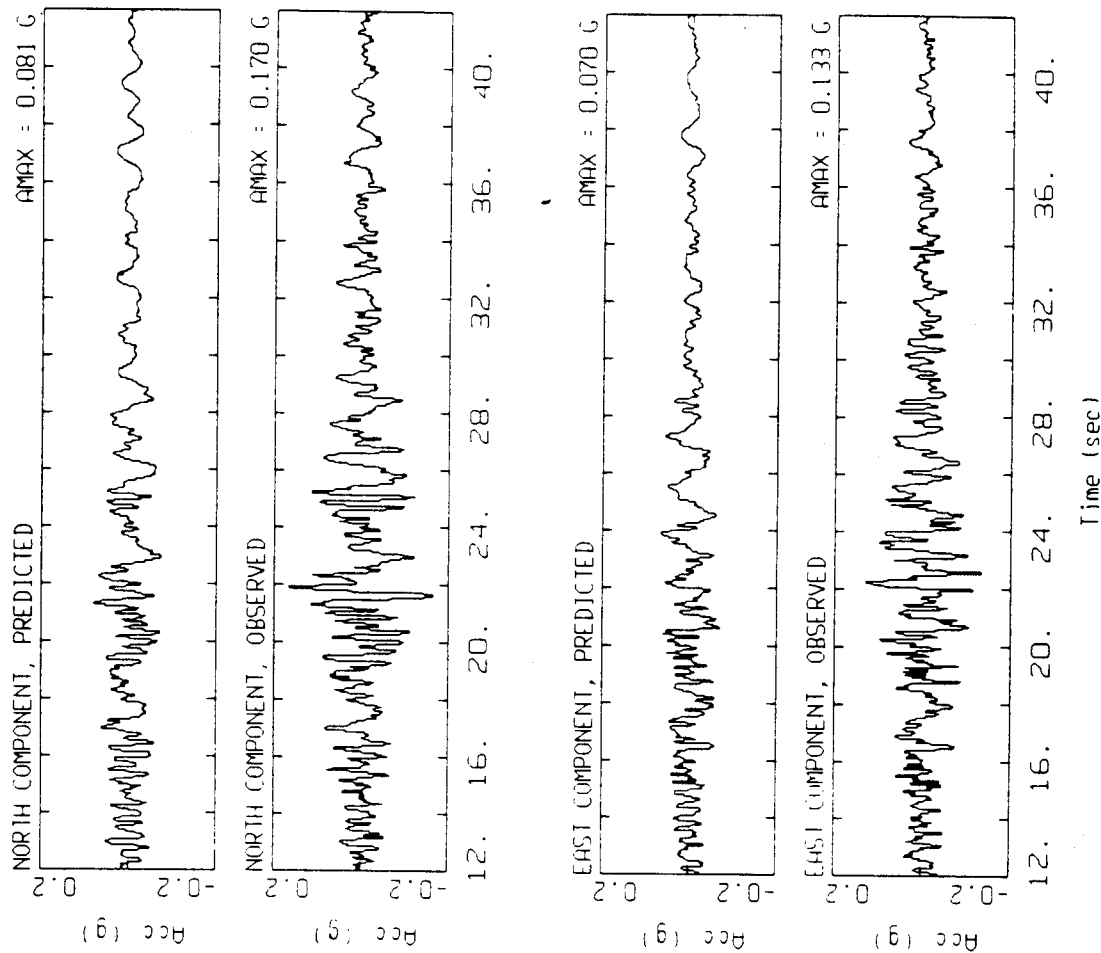
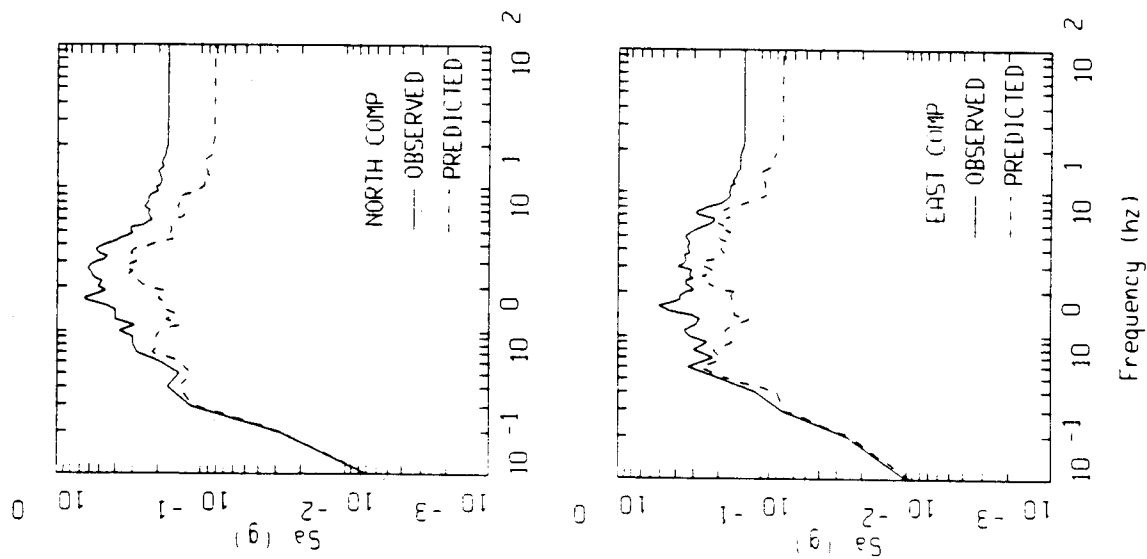


Figure 6.B-51

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for surface motions using the recordings at 154 ft as control motions. DESRA analysis.

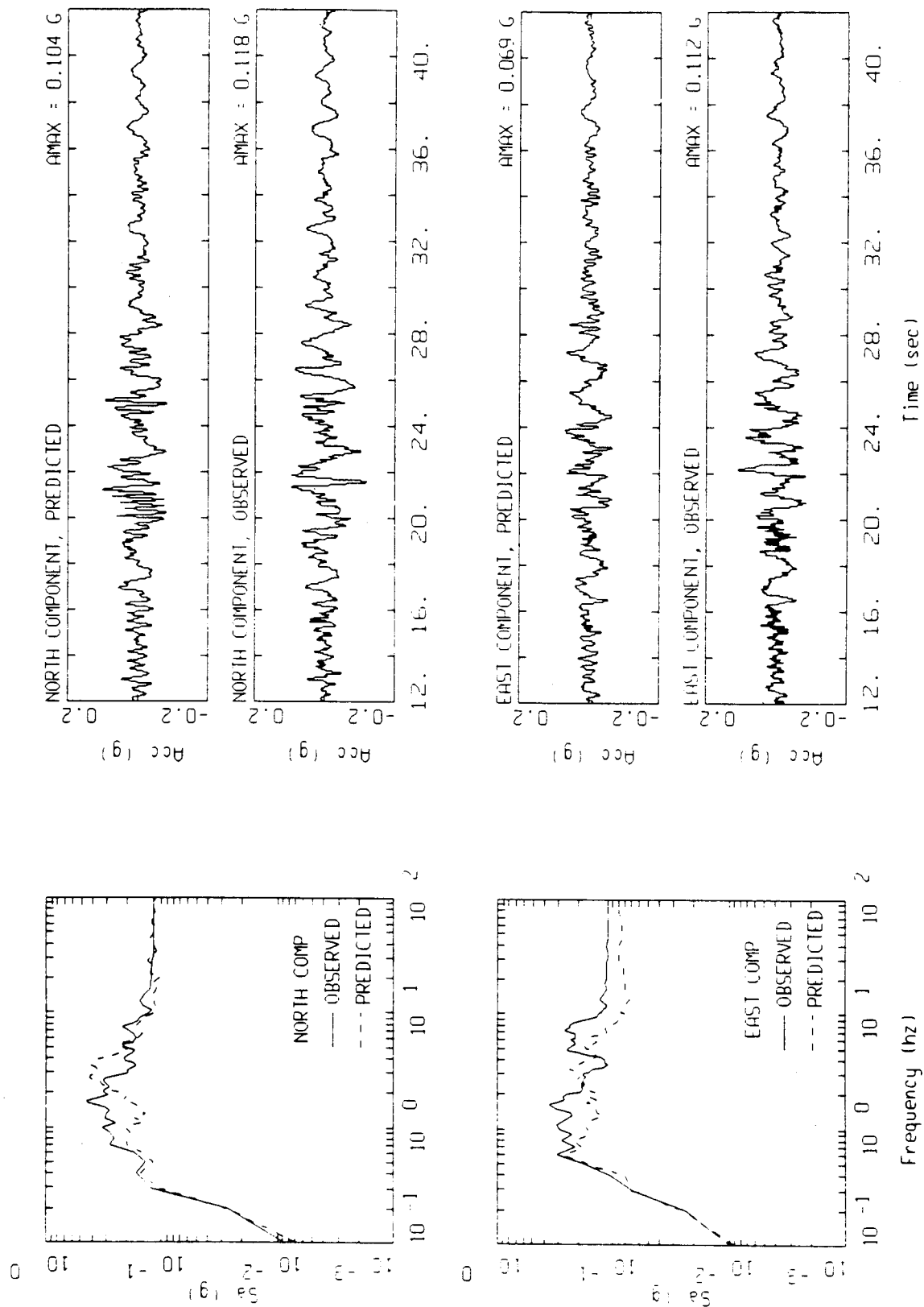


Figure 6.B-52

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for 20 ft depth using the recordings at 154 ft as control motions. DESRA analysis.

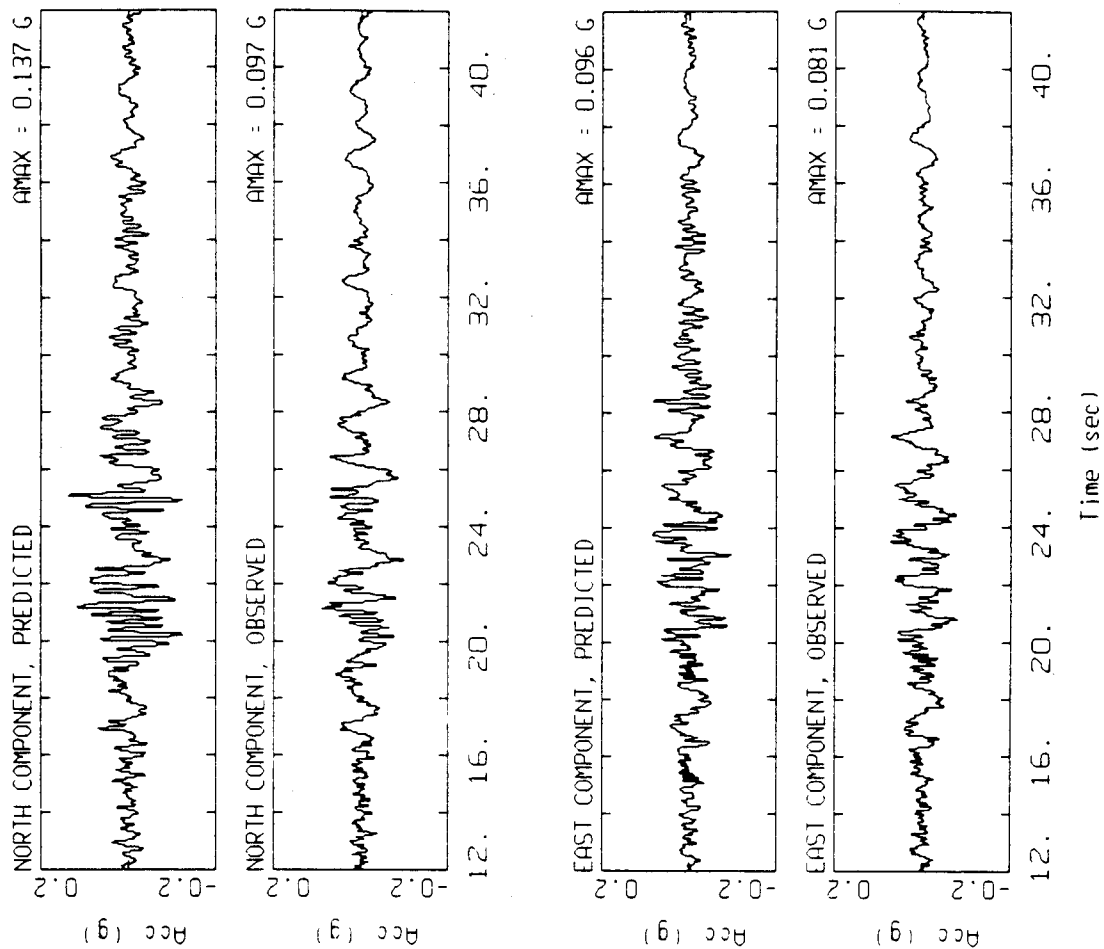
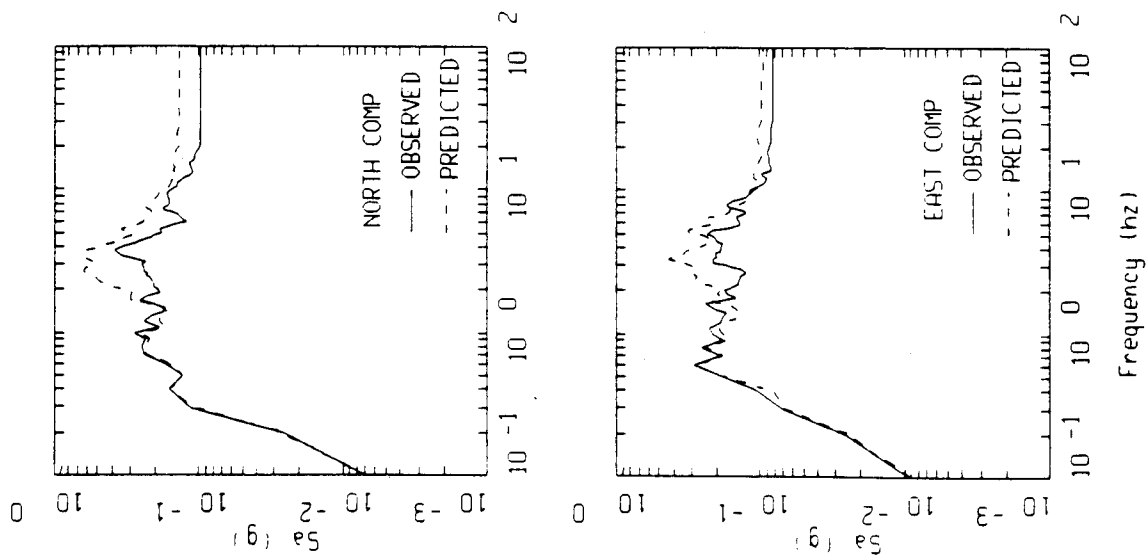


Figure 6.B-53

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for 36 ft depth using the recordings at 154 ft as control motions. DESRA analysis.

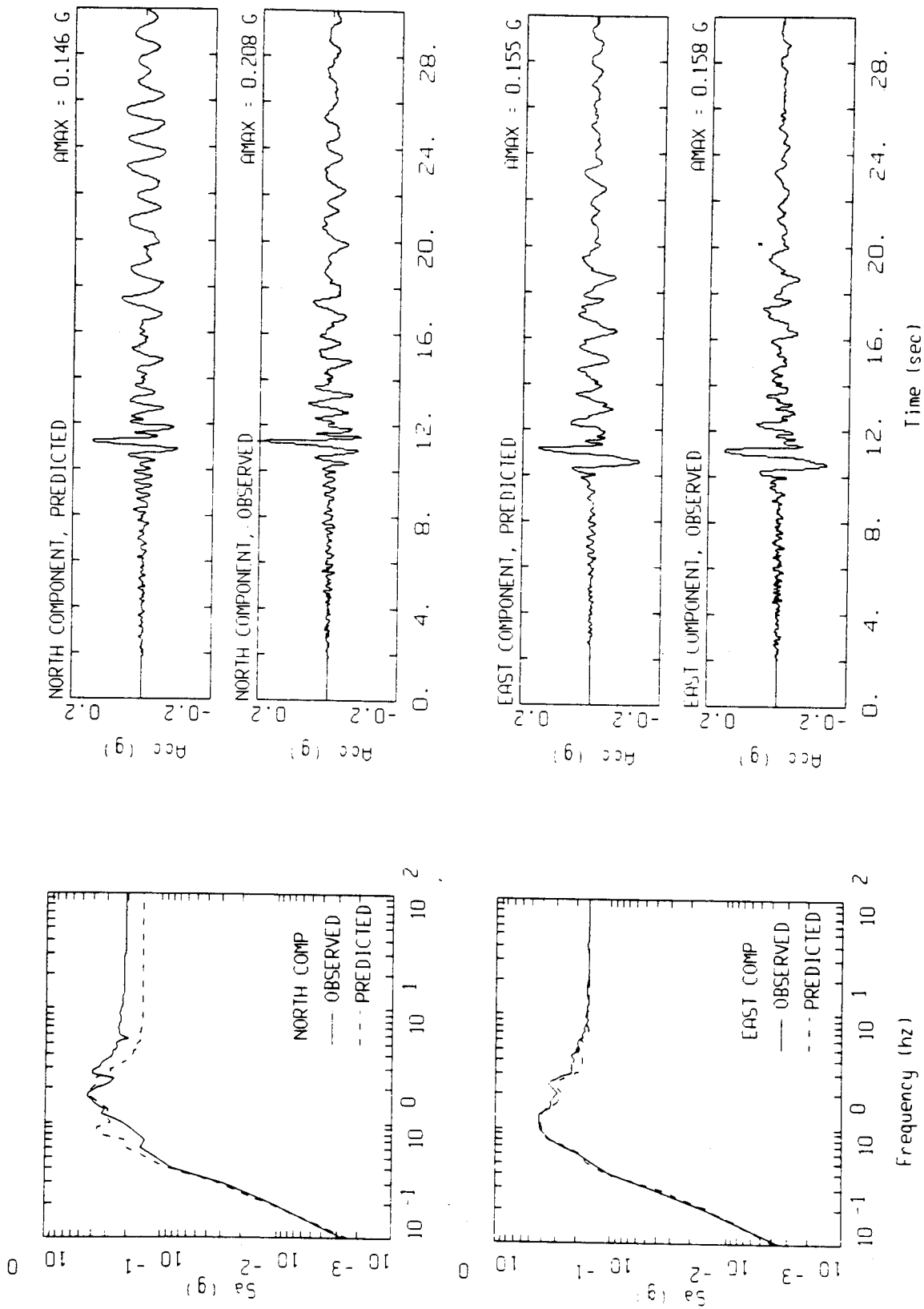


Figure 6.B-54

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for surface motions using the recordings at 154 ft as control motions. SUMDES analysis.

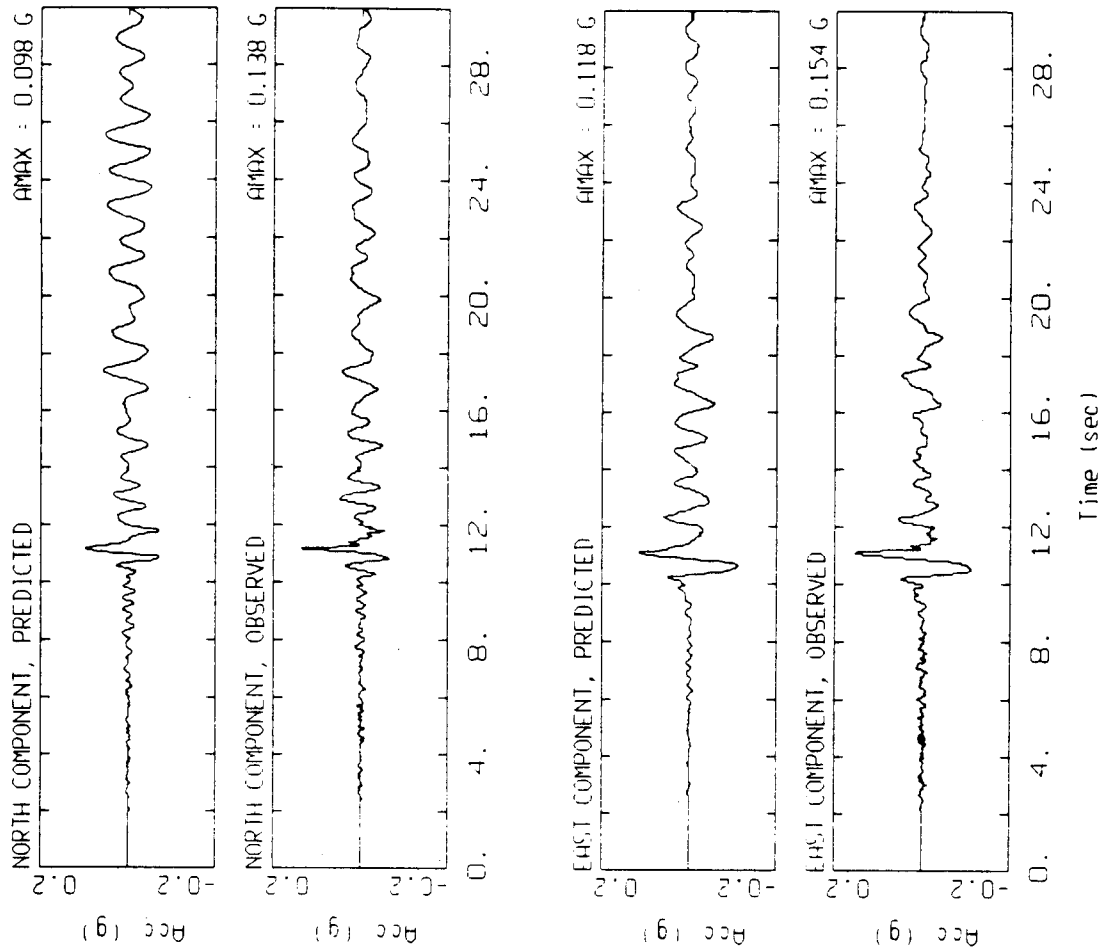
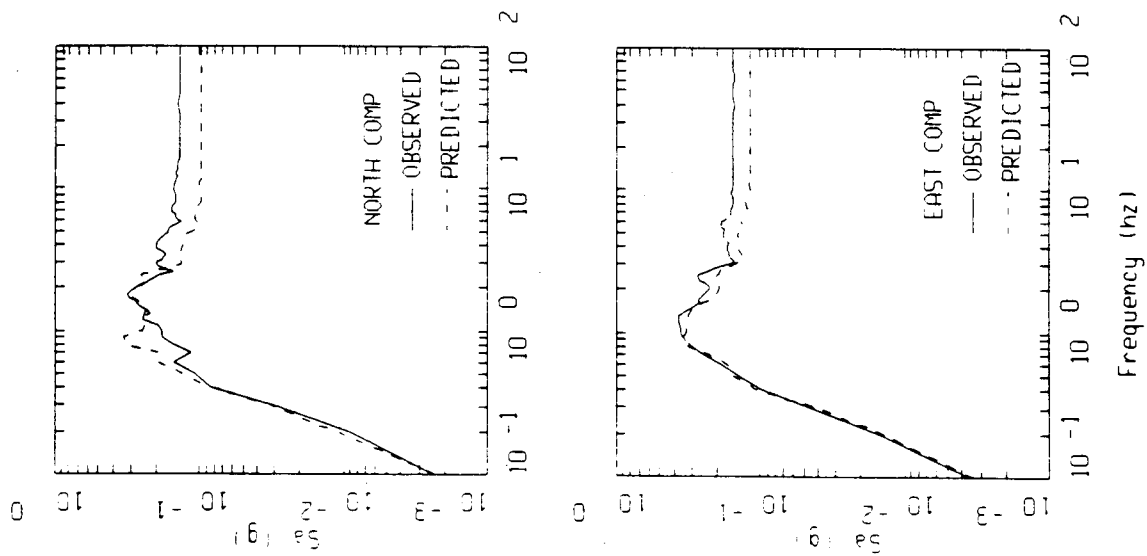


Figure 6.B-55

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 20 ft depth using the recordings at 154 ft as control motions. SUMDES analysis.

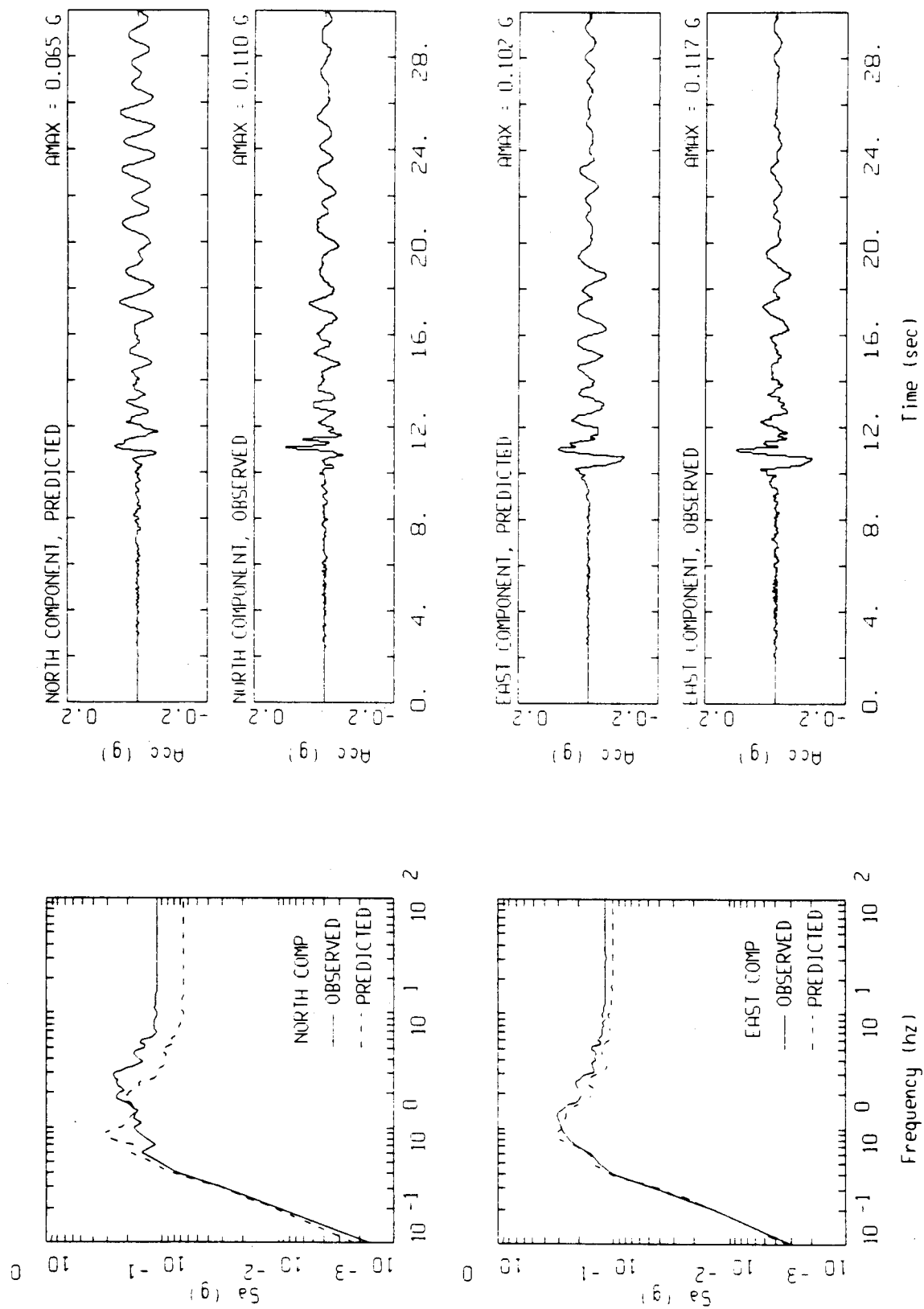


Figure 6.B-56
Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 36 ft depth using the recordings at 154 ft as control motions. SUMDES analysis.

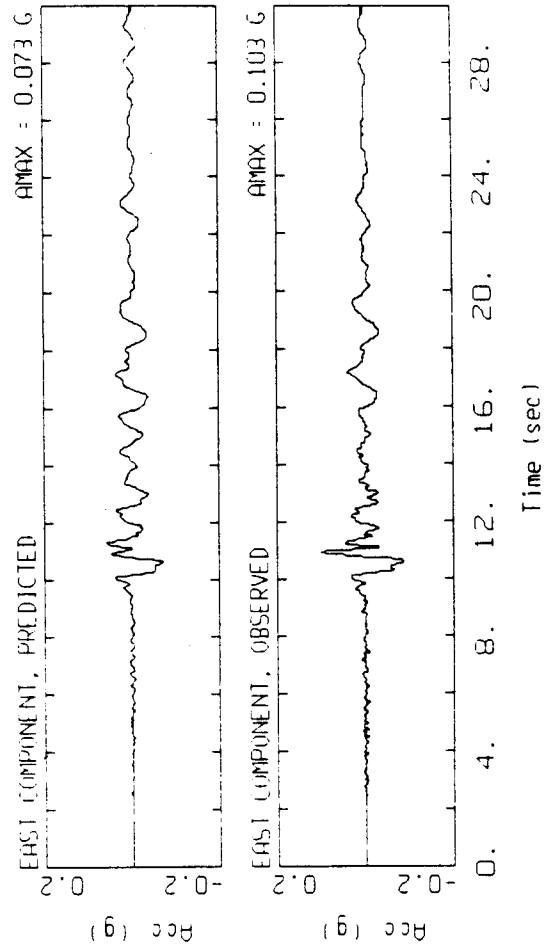
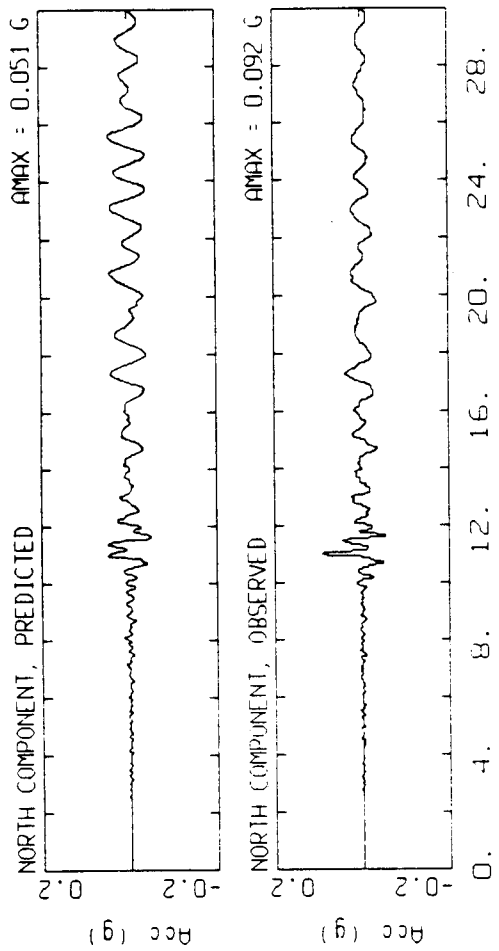
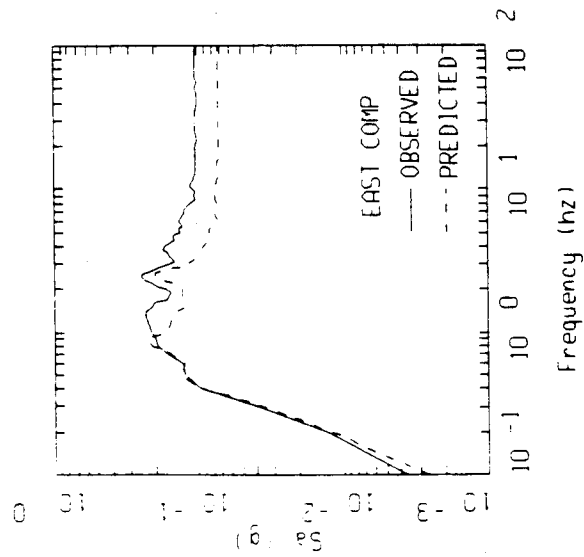
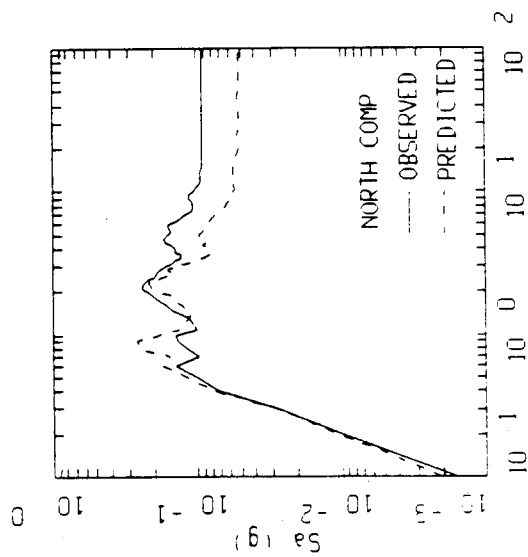


Figure 6.B-57
Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 56 ft depth using the recordings at 154 ft as control motions. SUMDES analysis.

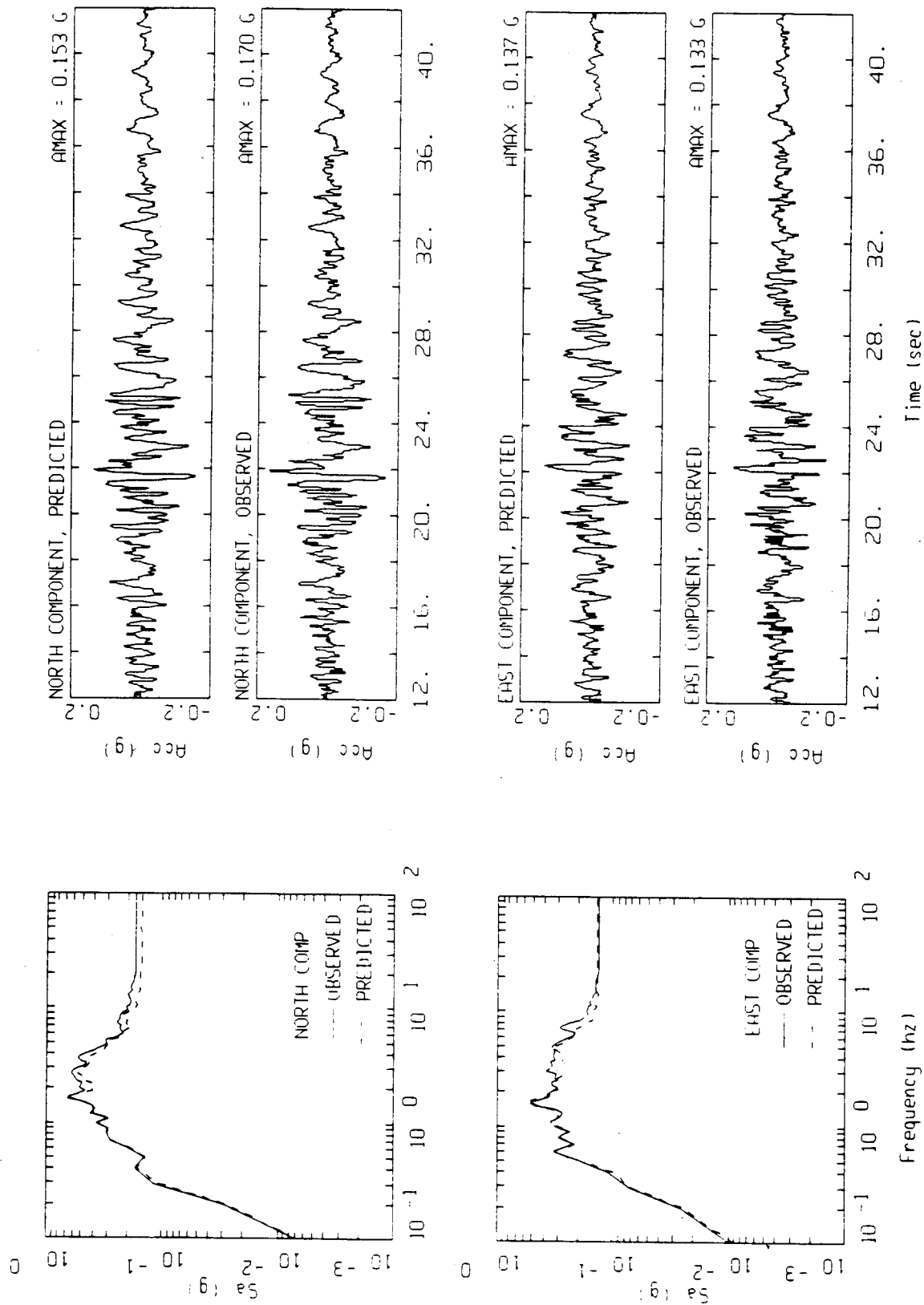


Figure 6.B-58

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for surface motions using the recordings at 154 ft as control motions. SUMDES analysis.

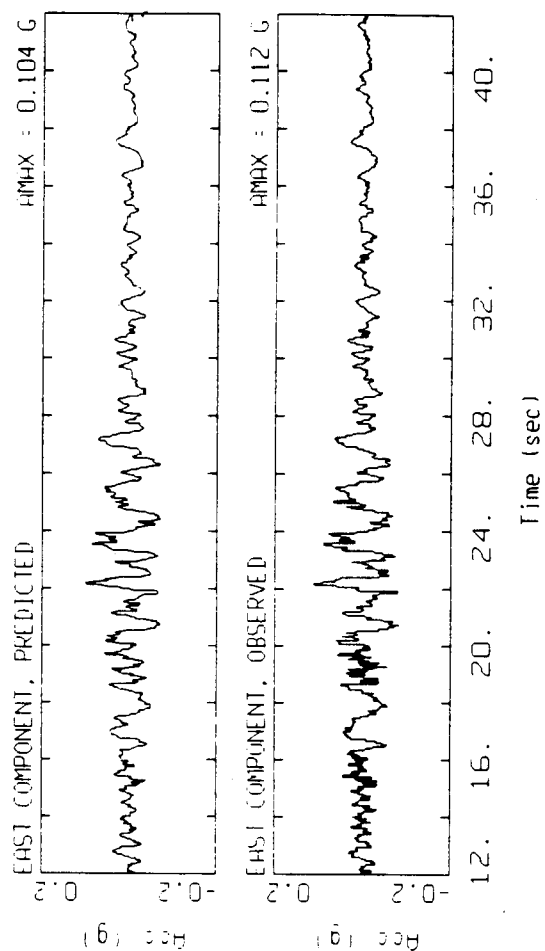
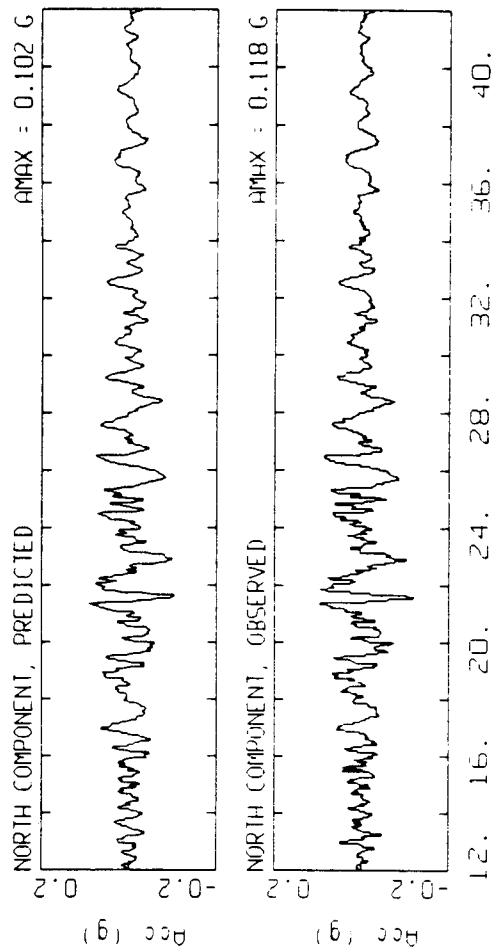
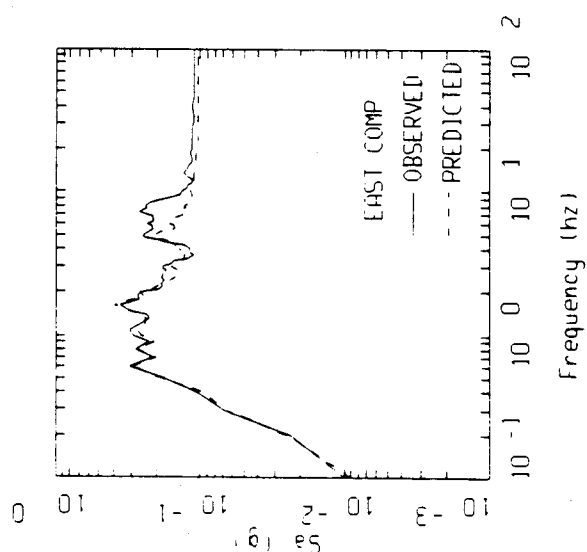
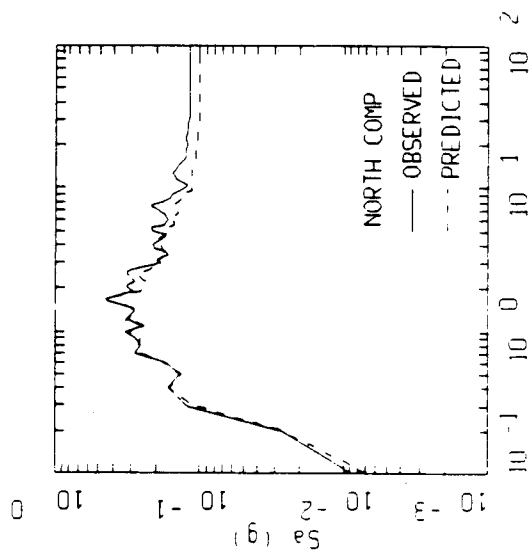


Figure 6.B-59

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for 20 ft depth using the recordings at 154 ft as control motions. SUMDES analysis.

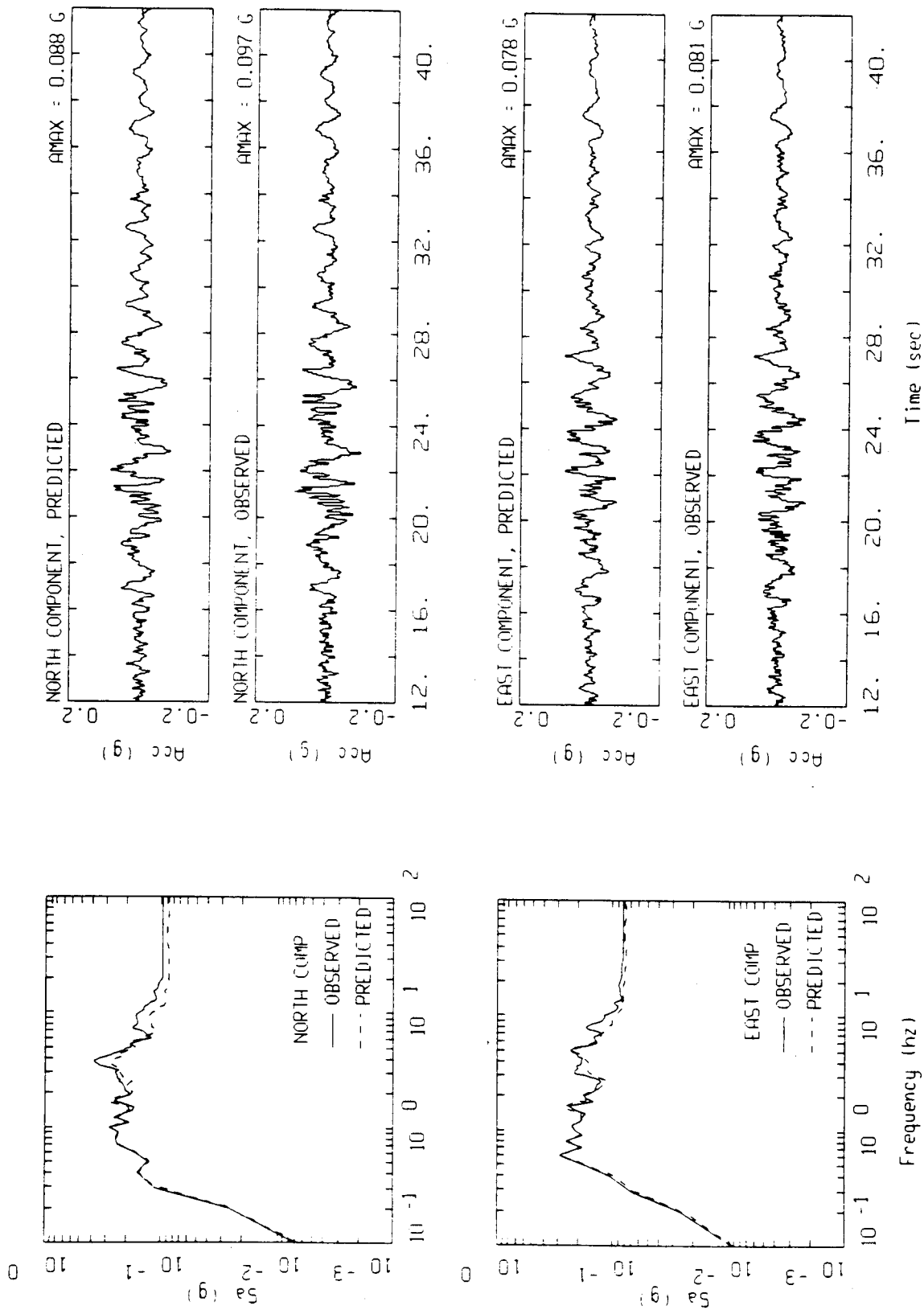


Figure 6.B-60

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for 36 ft depth using the recordings at 154 ft as control motions. SUMDES analysis.

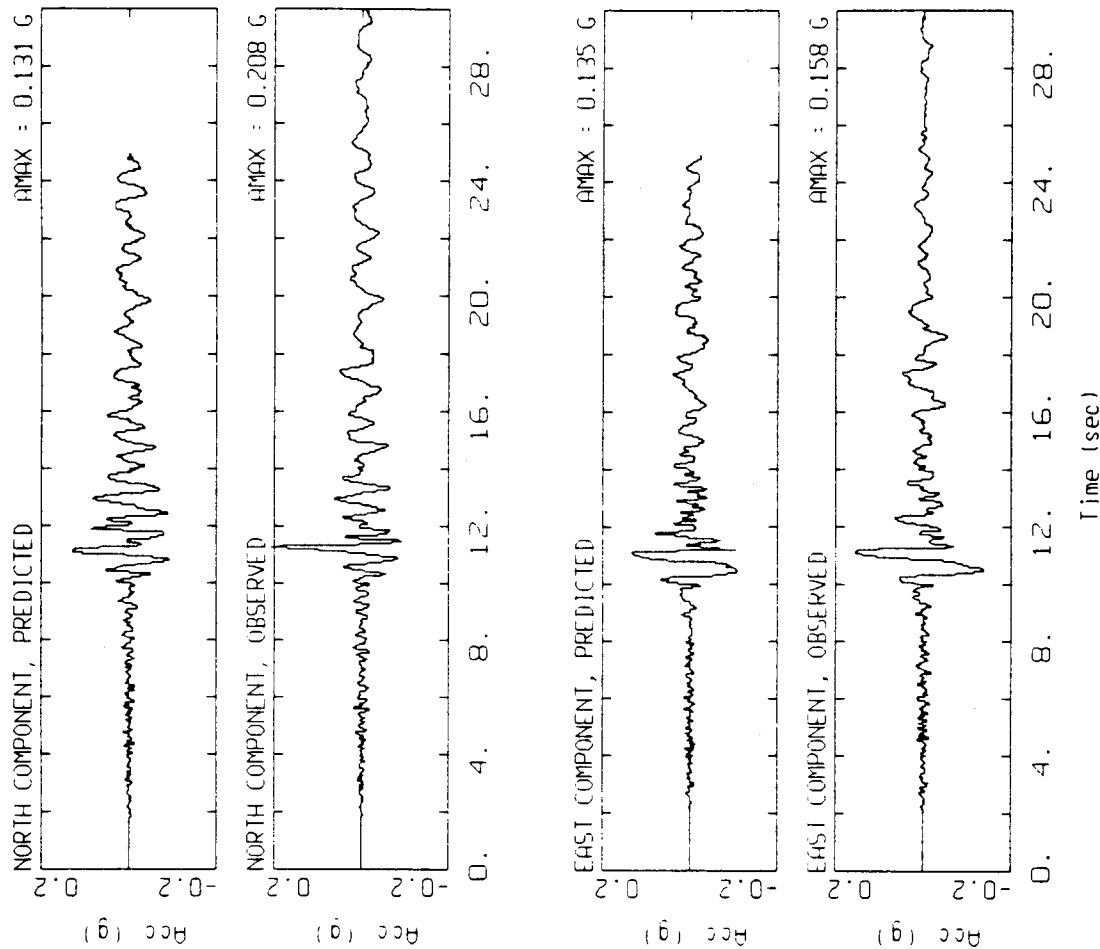
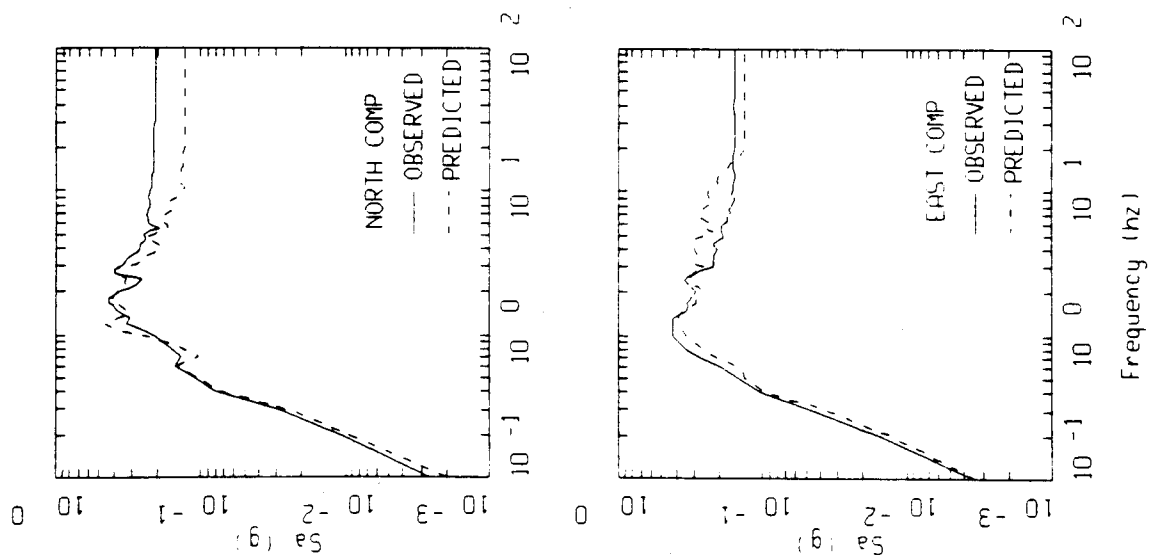


Figure 6.B-61

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for surface motions using the recordings at 154 ft as control motions. TESS analysis.

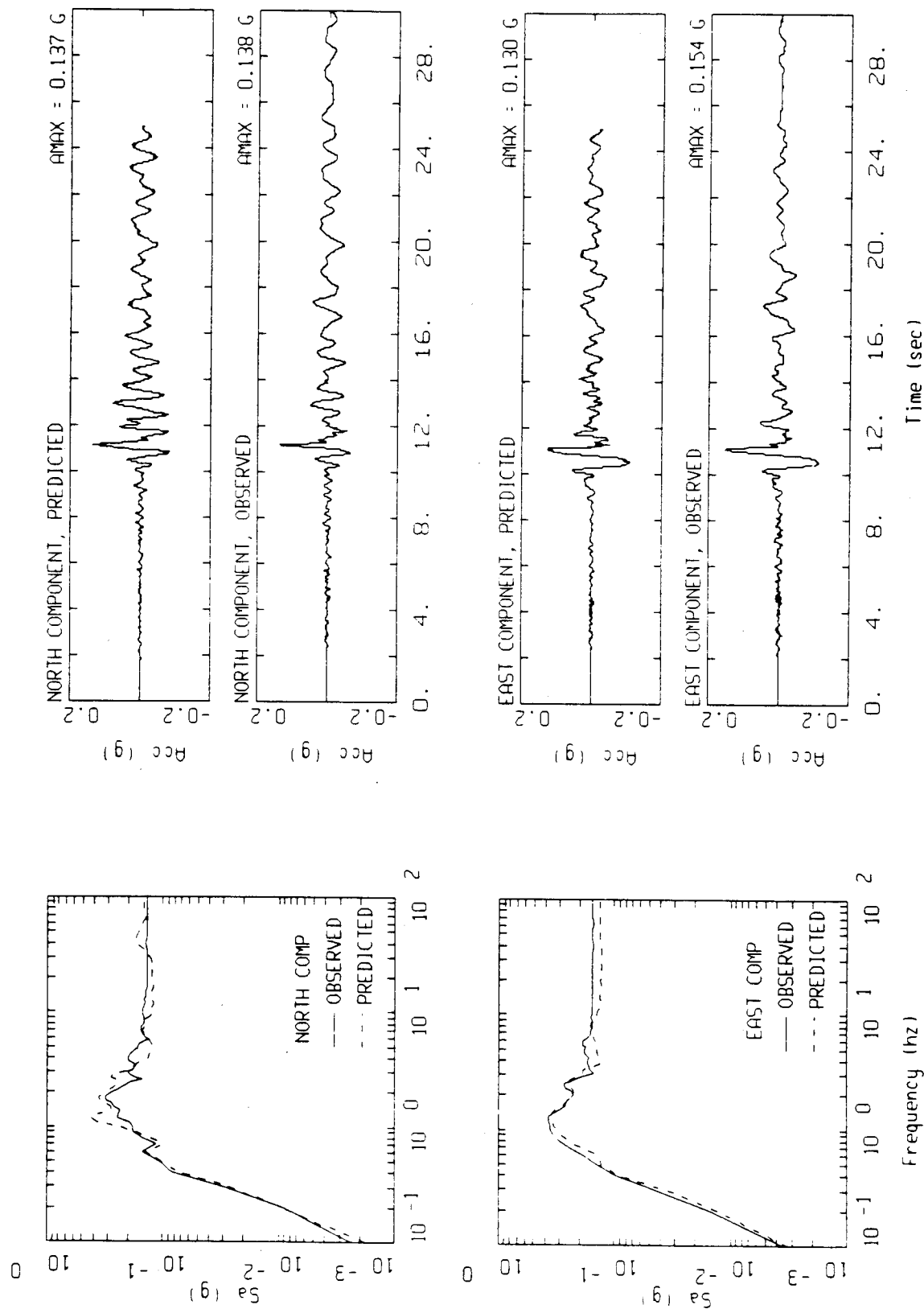


Figure 6.B-62
Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 20 ft depth using the recordings at 154 ft as control motions. TESS analysis.

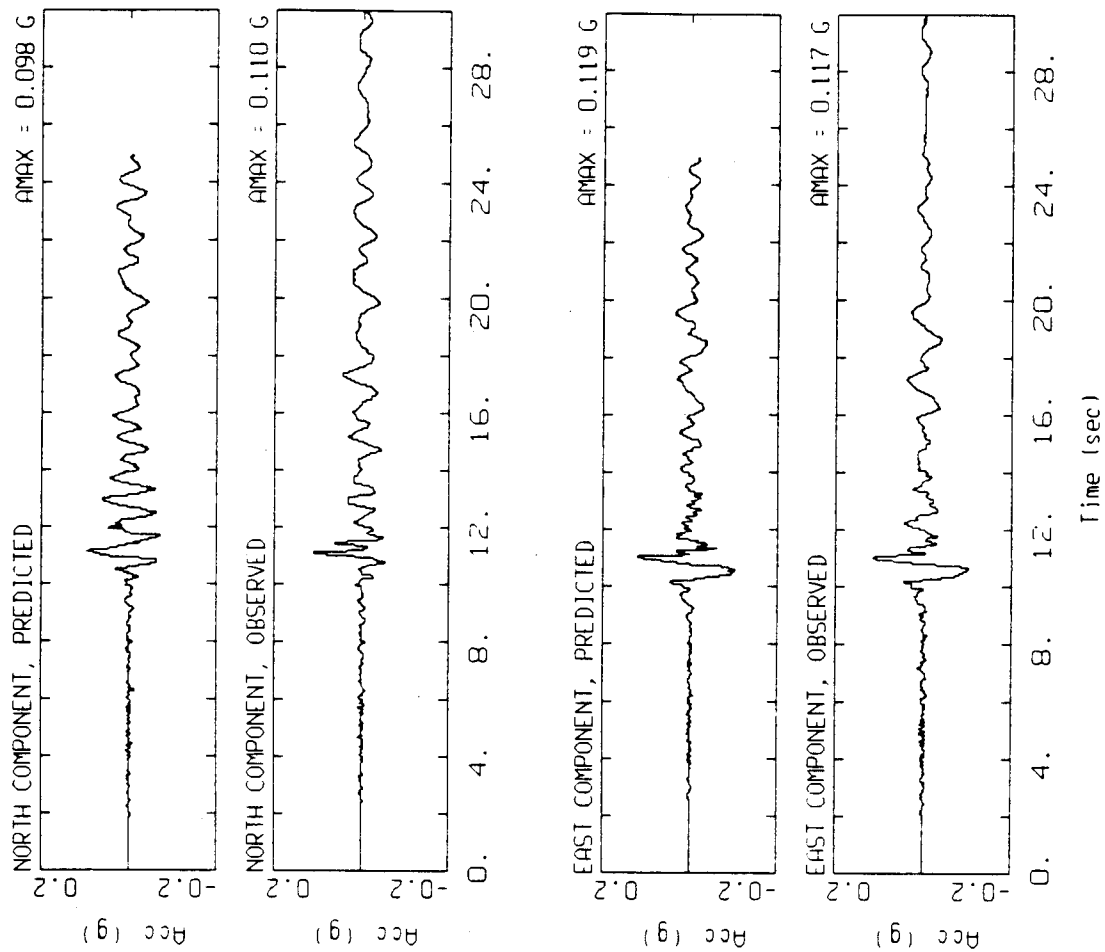
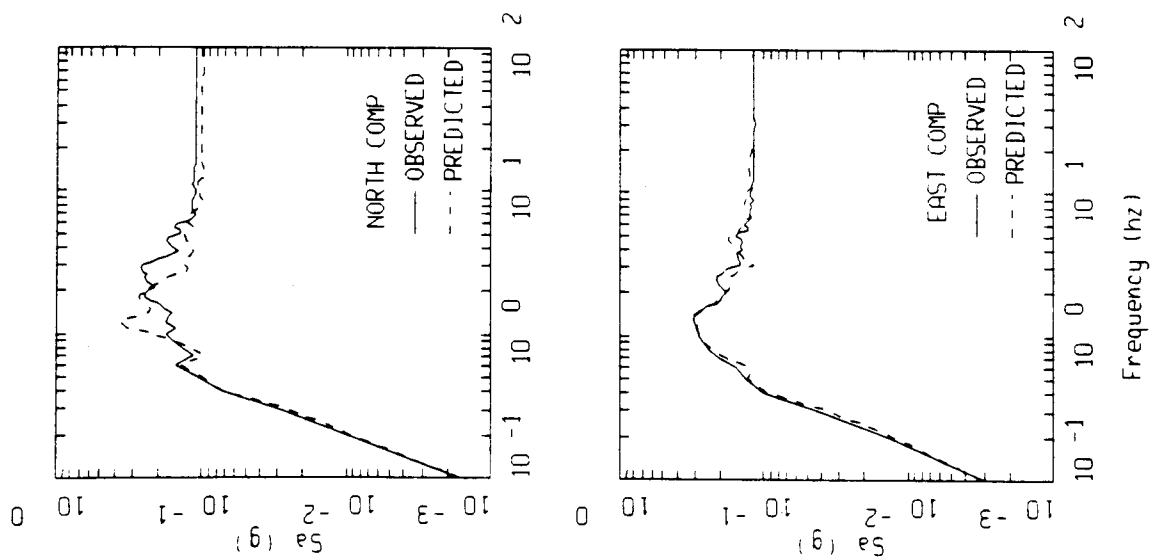


Figure 6.B-63
Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 36 ft depth using the recordings at 154 ft as control motions. TESS analysis.

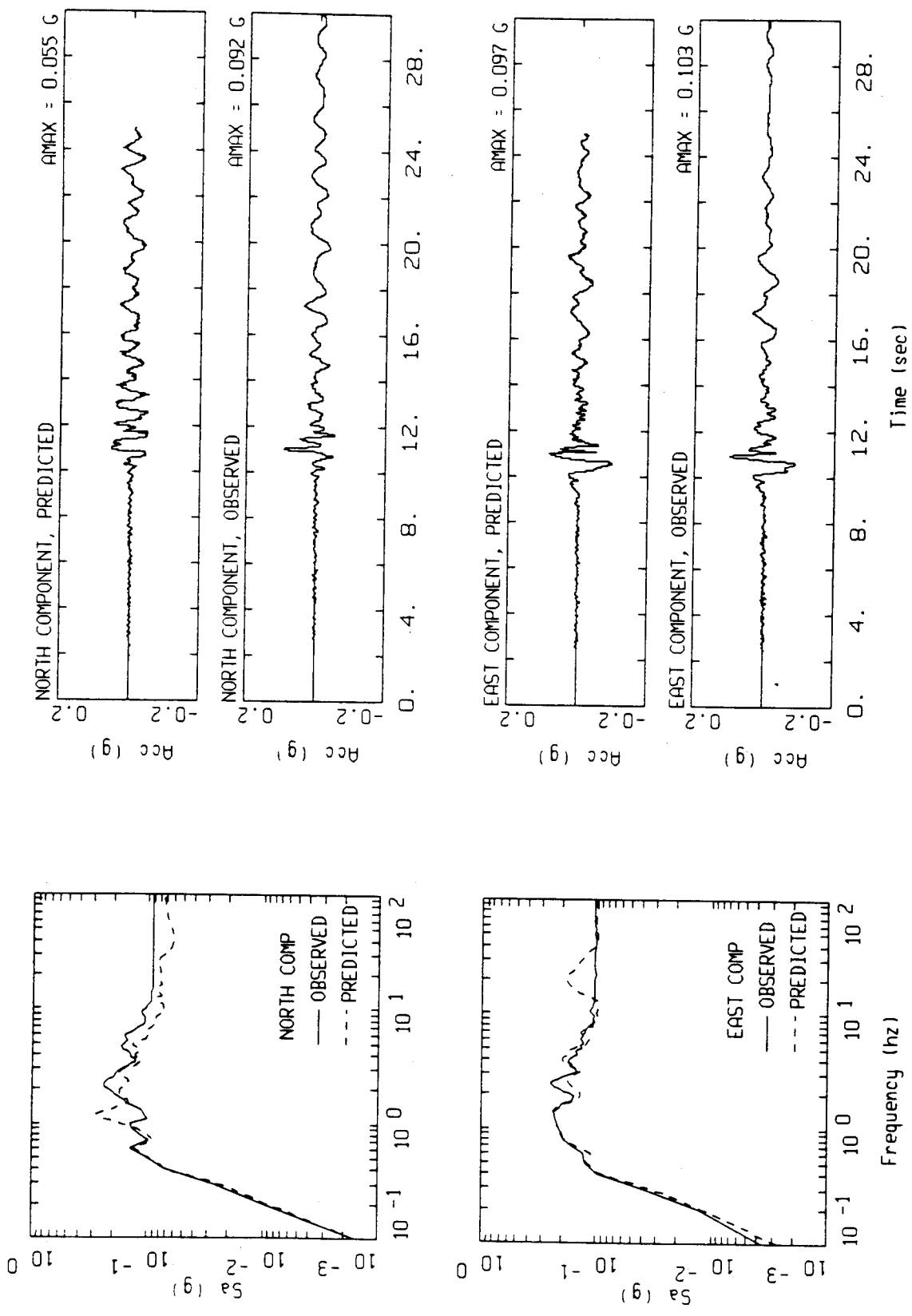


Figure 6.B-64

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 7 at reference site Lotung for 56 ft depth using the recordings at 154 ft as control motions. TESS analysis.

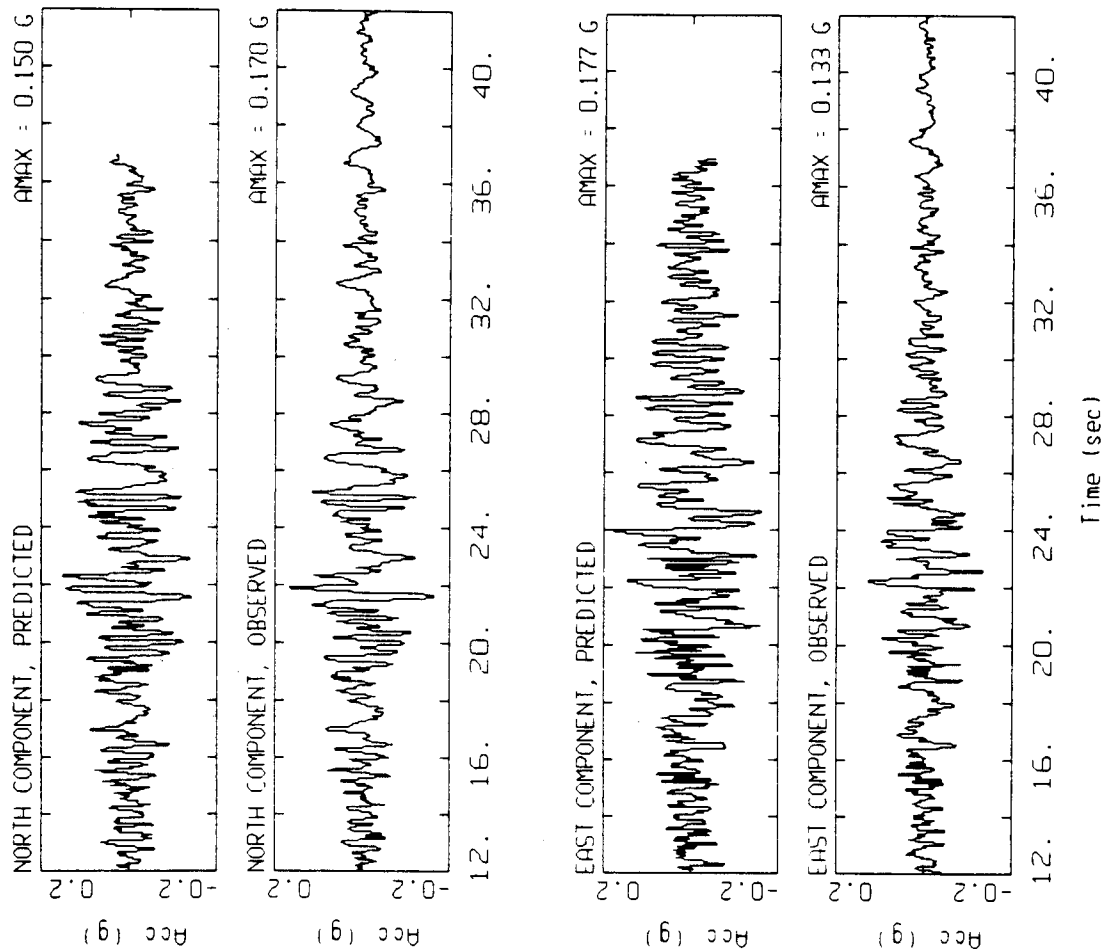
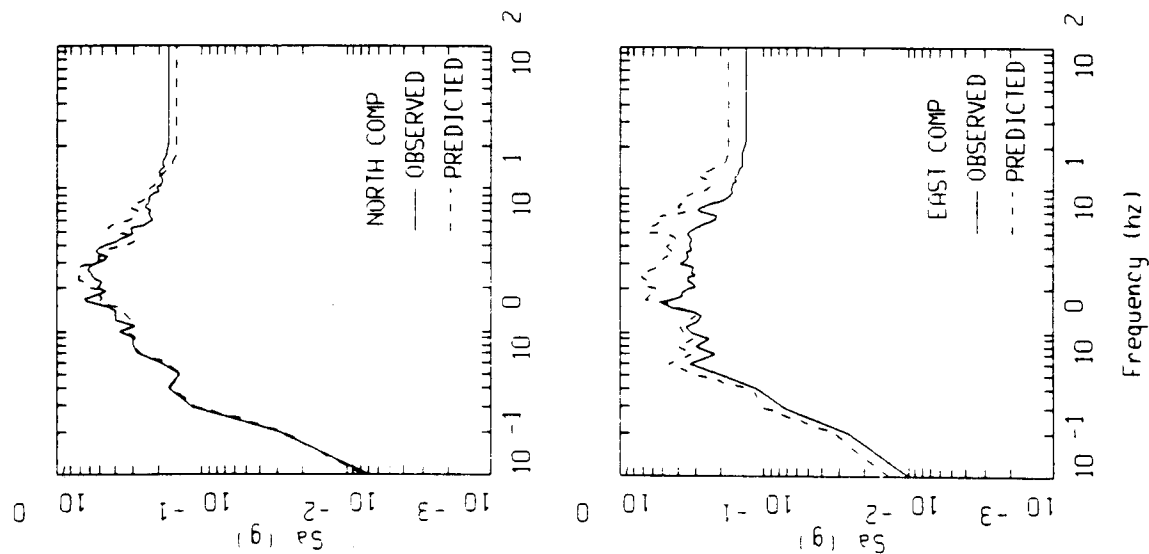


Figure 6.B-65

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for surface motions using the recordings at 154 ft as control motions. TESS analysis.

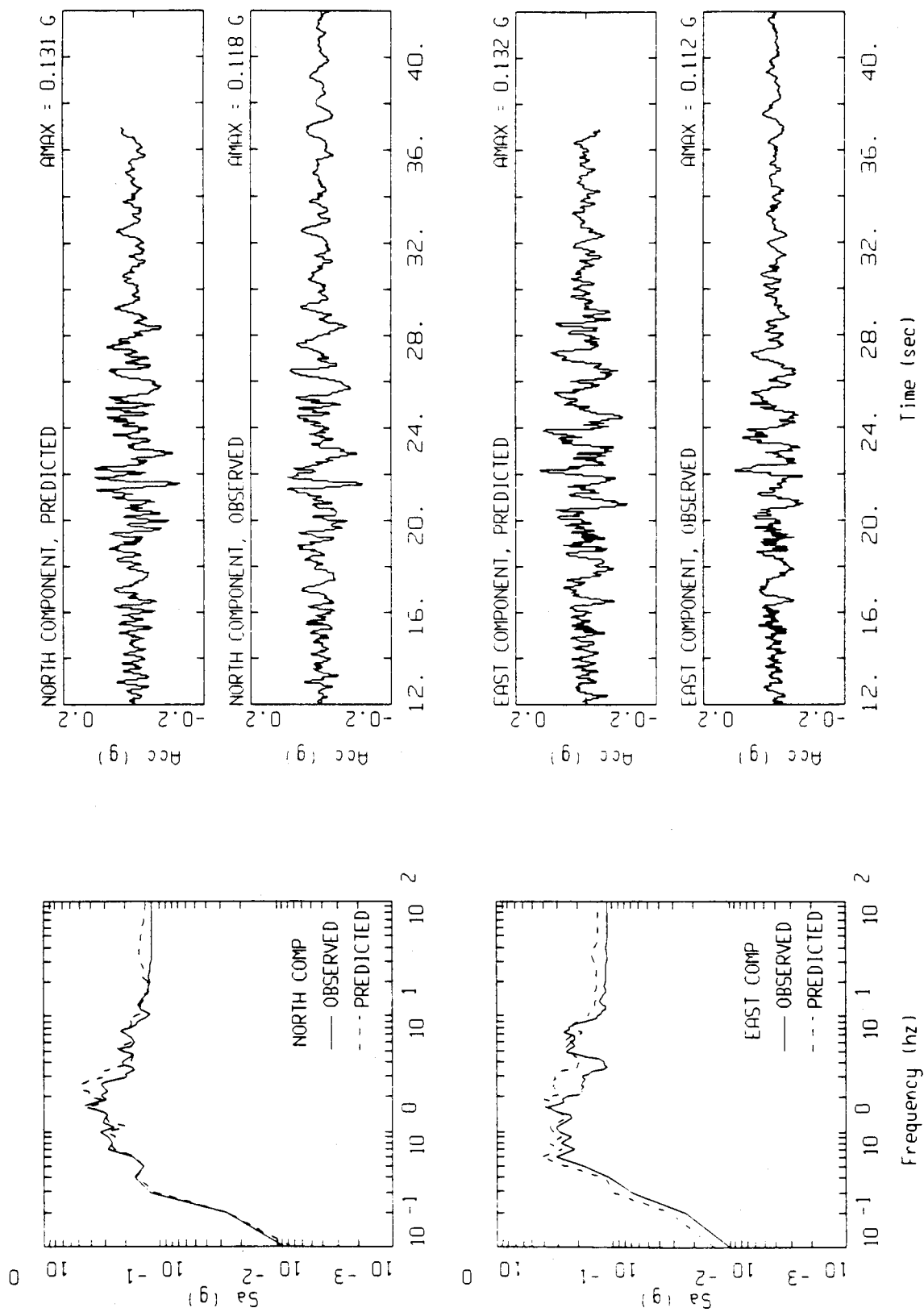


Figure 6.B-66

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for 20 ft depth using the recordings at 154 ft as control motions. TESS analysis.

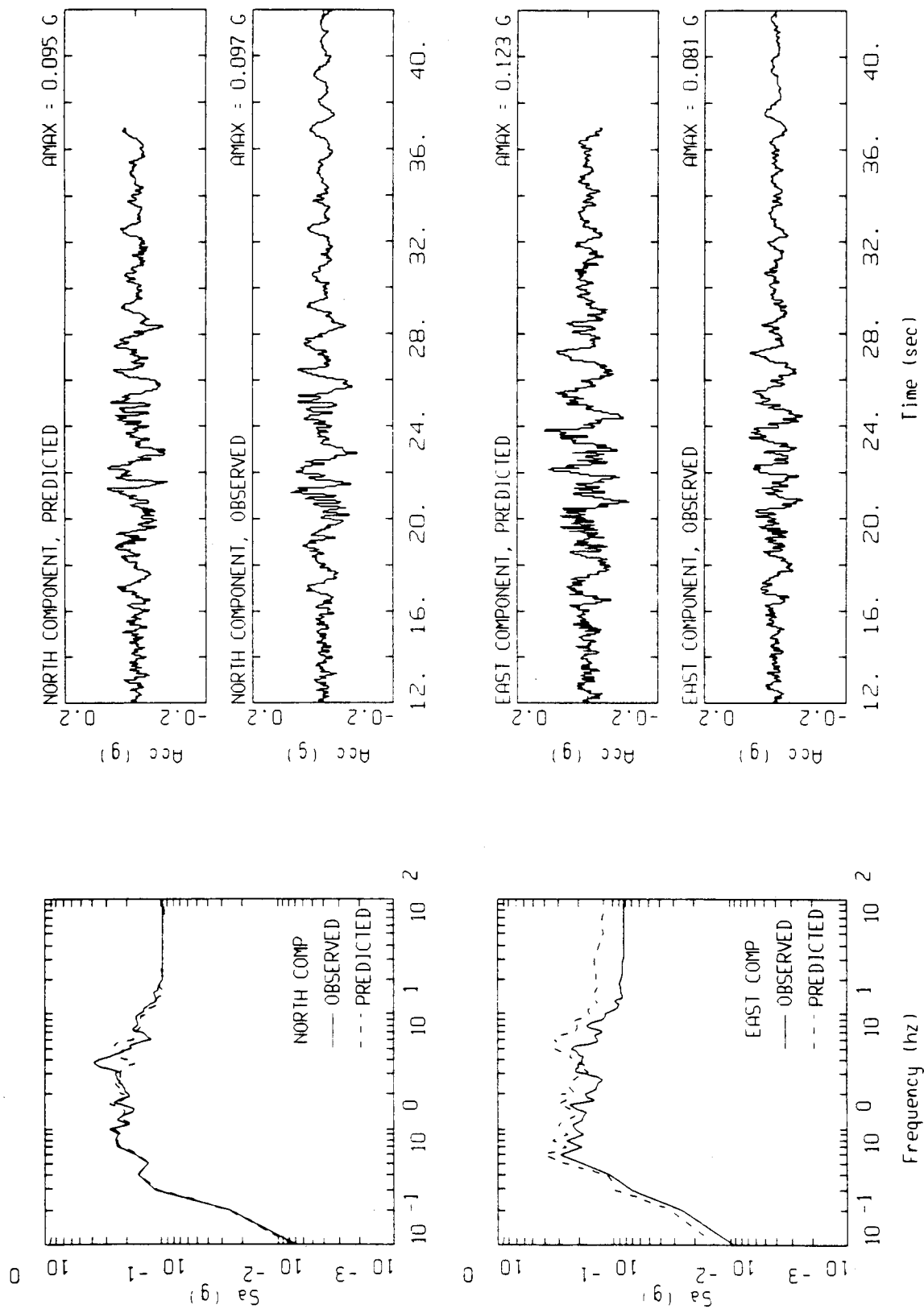


Figure 6.B-67

Comparison of observed and predicted time histories and 5% damped spectral accelerations for LSST event 16 at reference site Lotung for 36 ft depth using the recordings at 154 ft as control motions. TESS analysis.

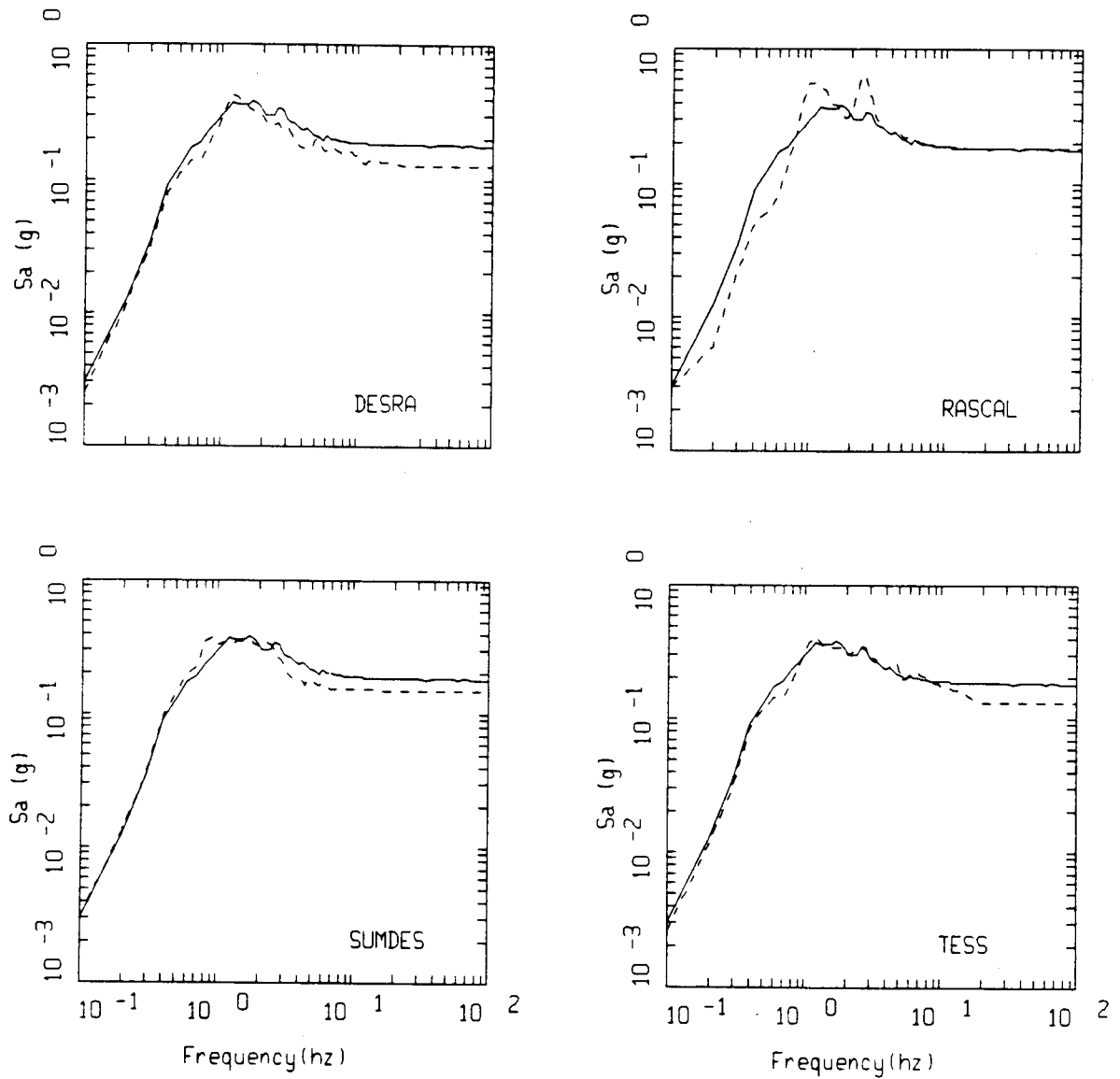


Figure 6.B-68

Comparison of log average (horizontal components) 5% damped spectral accelerations for the four analyses to the empirical spectra at reference site Lotung for LSST event 7 surface motions (empirical, solid; predicted, dashed).

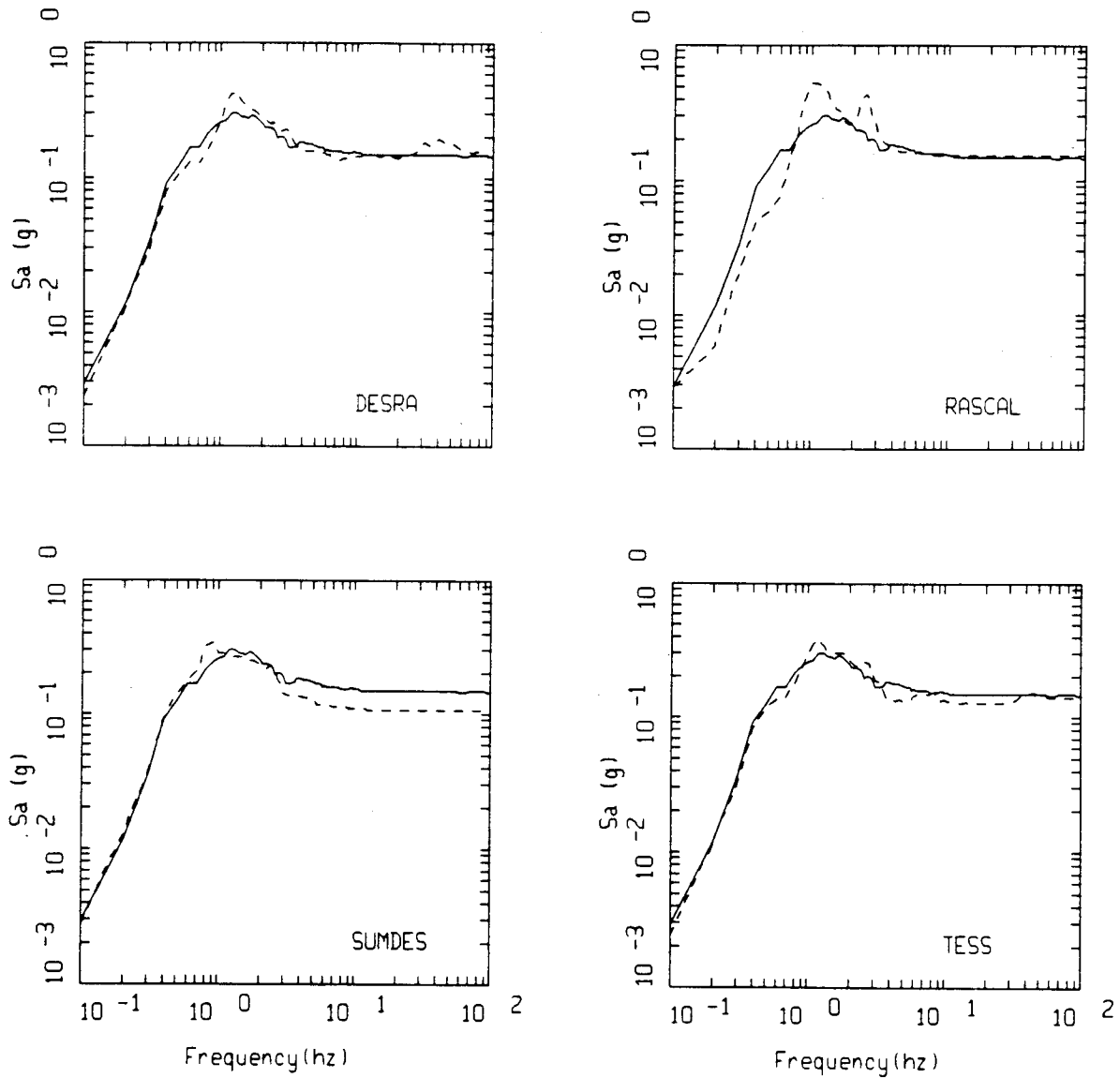


Figure 6.B-69

Comparison of log average (horizontal components) 5% damped spectral accelerations for the four analyses to the empirical spectra at reference site Lotung for LSST event 7 motions at 20 ft depth (empirical, solid; predicted, dashed).

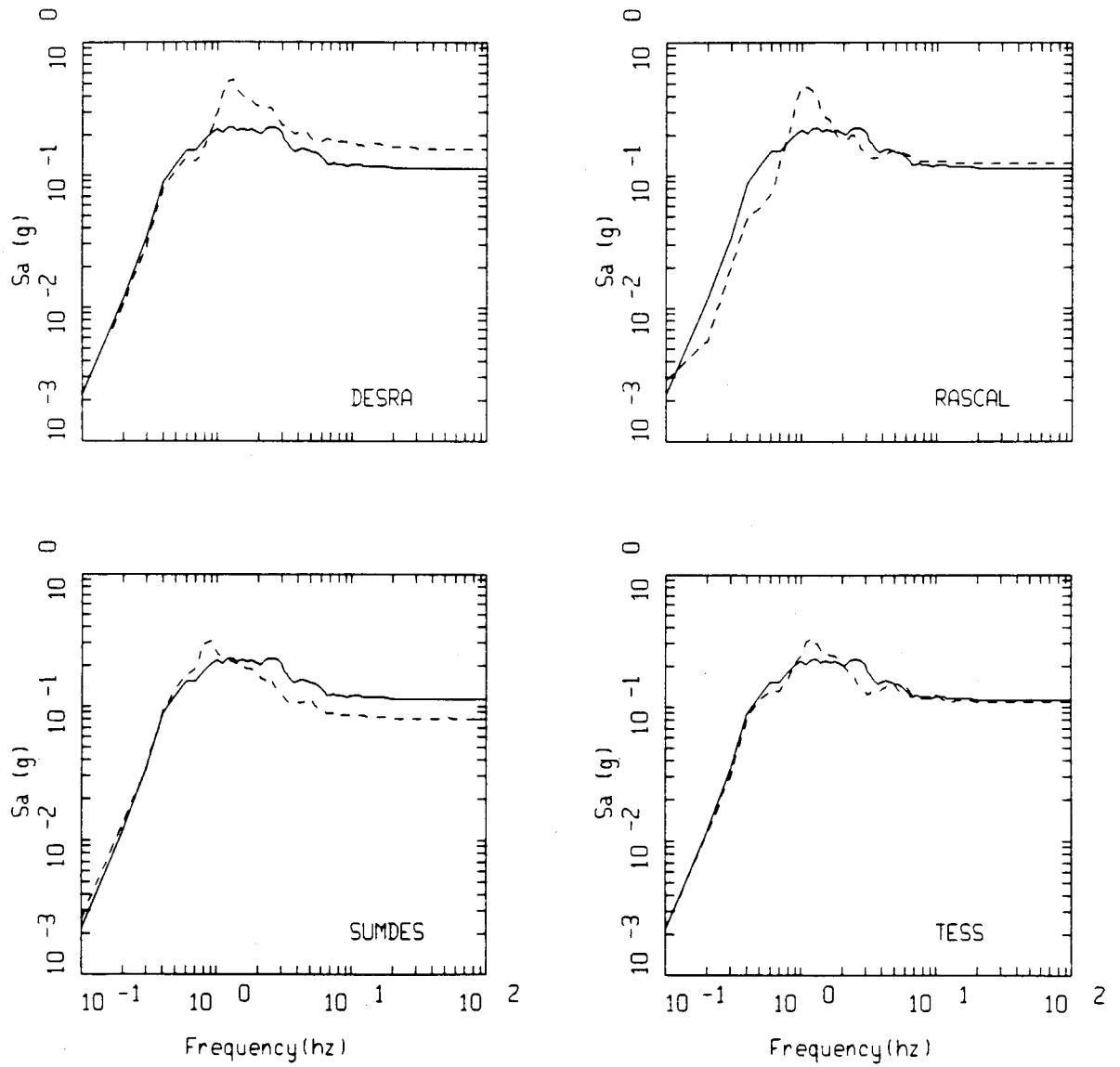


Figure 6.B-70

Comparison of log average (horizontal components) 5% damped spectral accelerations for the four analyses to the empirical spectra at reference site Lotung for LSST event 7 motions at 36 ft depth (empirical, solid; predicted, dashed).

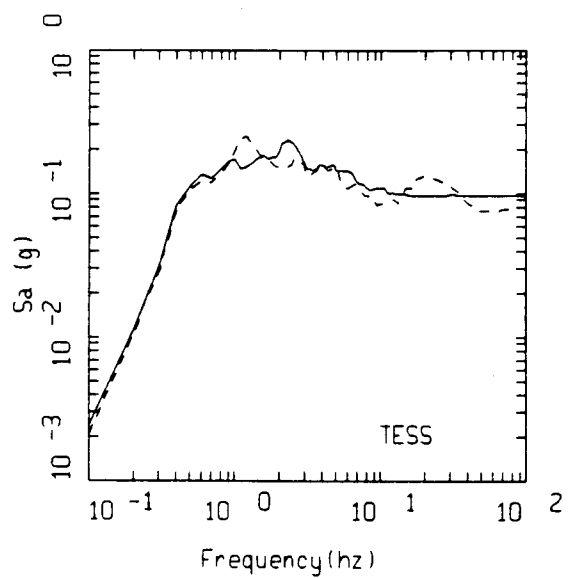
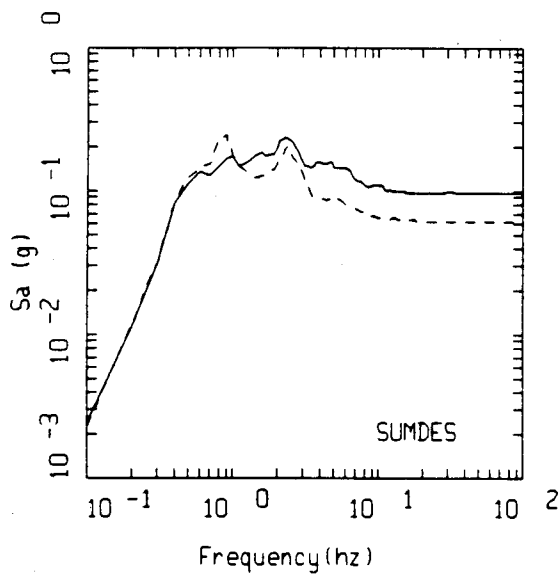
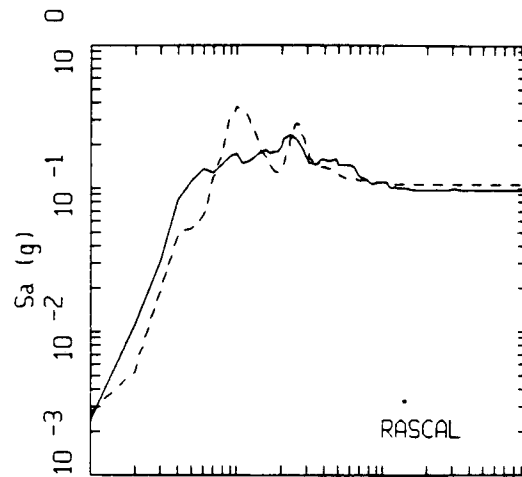
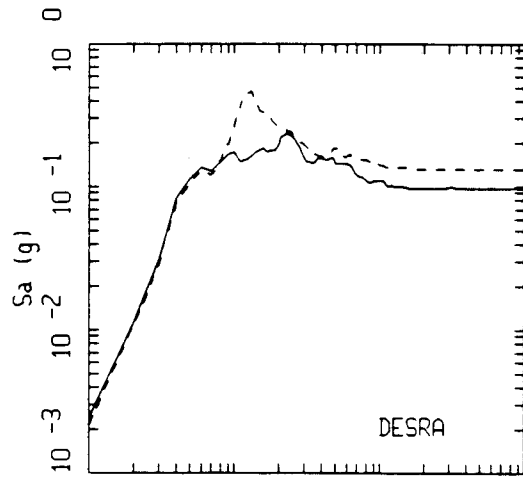


Figure 6.B-71

Comparison of log average (horizontal components) 5% damped spectral accelerations for the four analyses to the empirical spectra at reference site Lotung for LSST event 7 motions at 56 ft depth (empirical, solid; predicted, dashed).

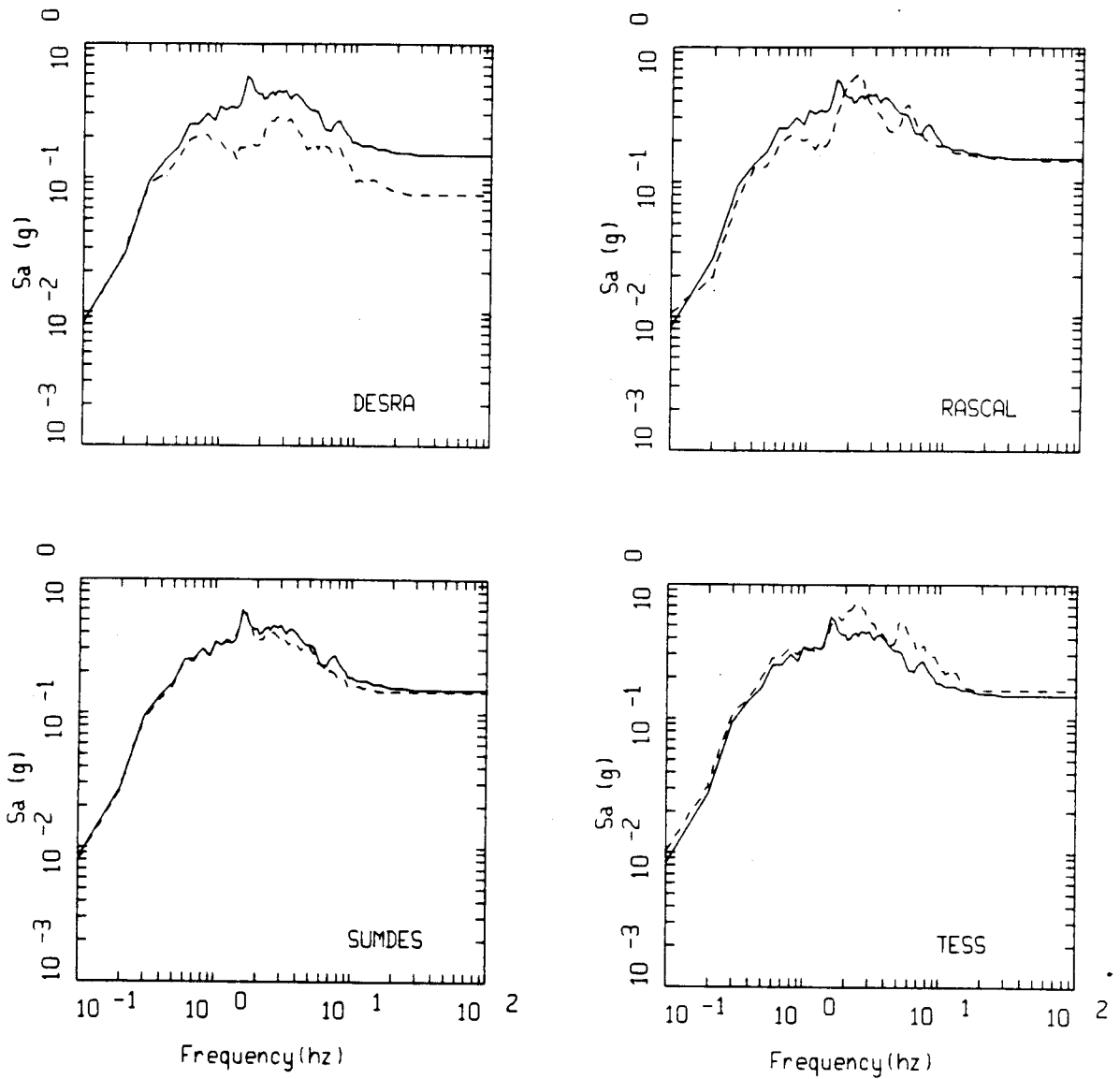


Figure 6.B-72

Comparison of log average (horizontal components) 5% damped spectral accelerations for the four analyses to the empirical spectra at reference site Lotung for LSST event 16 surface motions (empirical, solid; predicted, dashed).

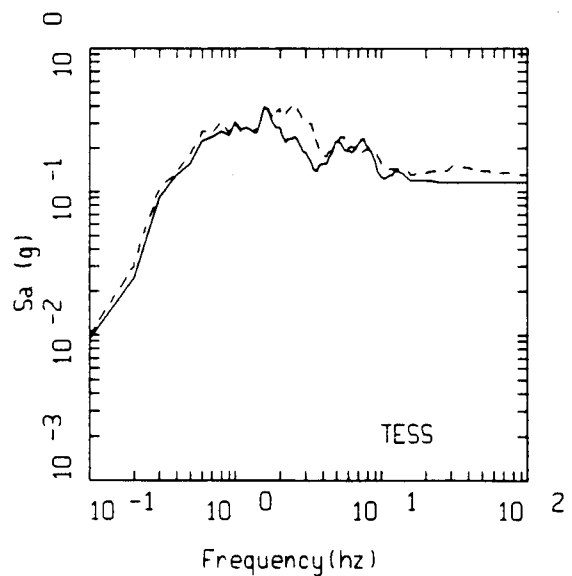
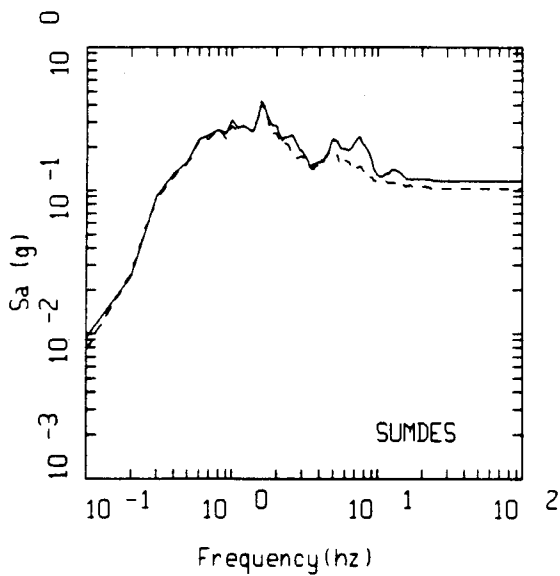
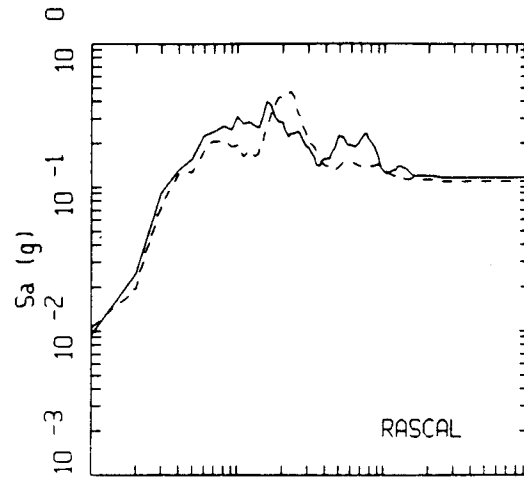
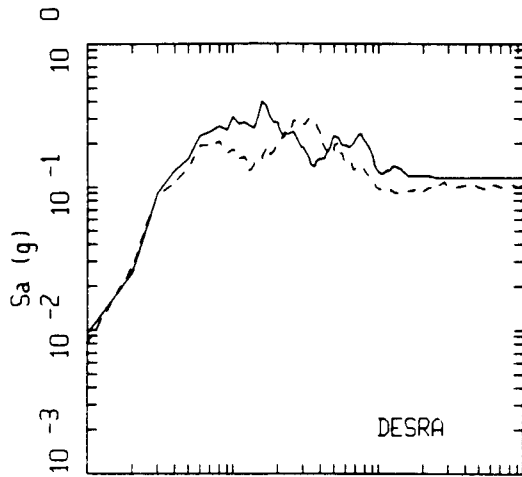


Figure 6.B-73

Comparison of log average (horizontal components) 5% damped spectral accelerations for the four analyses to the empirical spectra at reference site Lotung for LSST event 16 motions at 20 ft depth (empirical, solid; predicted, dashed).

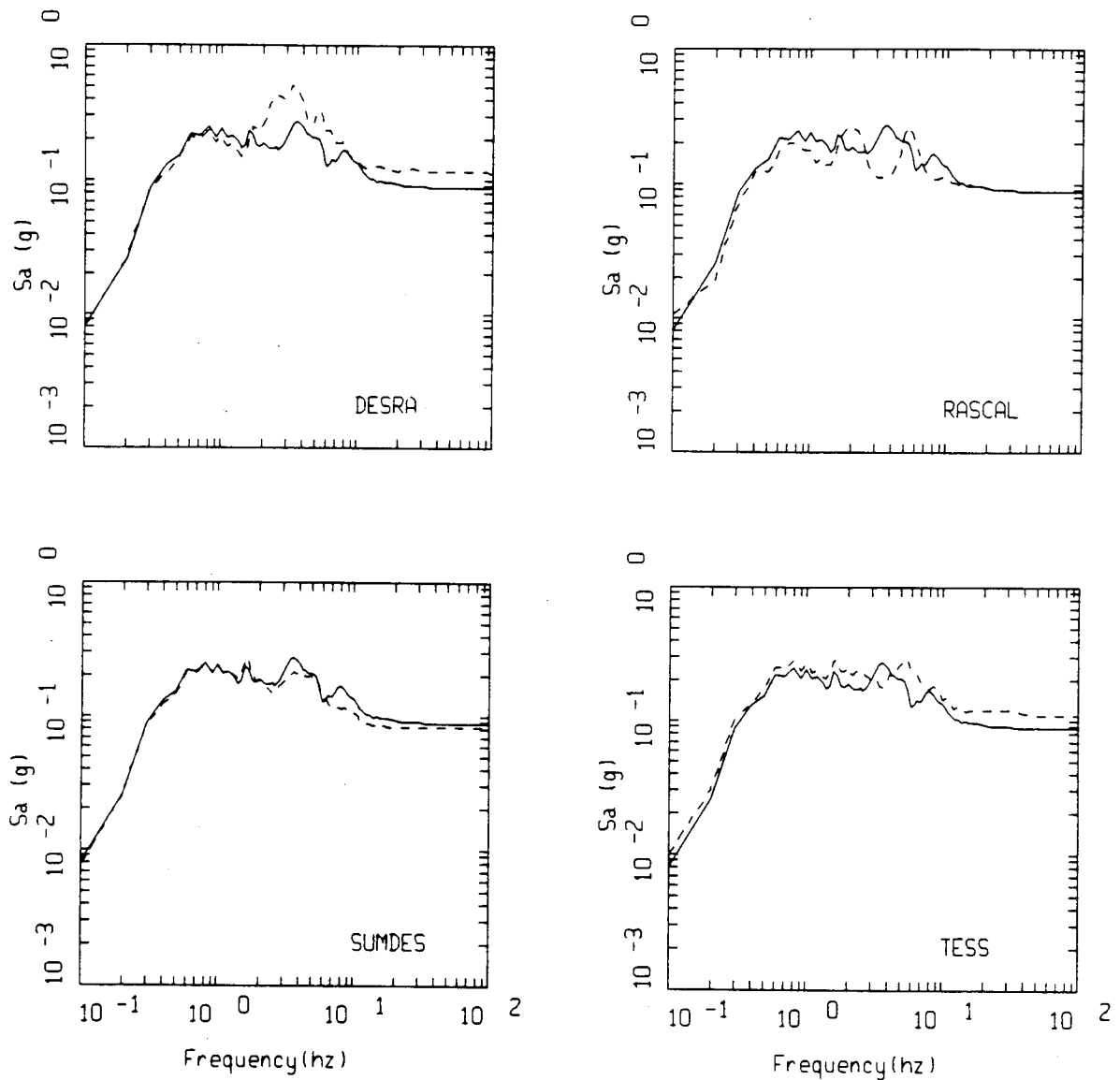


Figure 6.B-74

Comparison of log average (horizontal components) 5% damped spectral accelerations for the four analyses to the empirical spectra at reference site Lotung for LSST event 16 motions at 36 ft depth (empirical, solid; predicted, dashed).

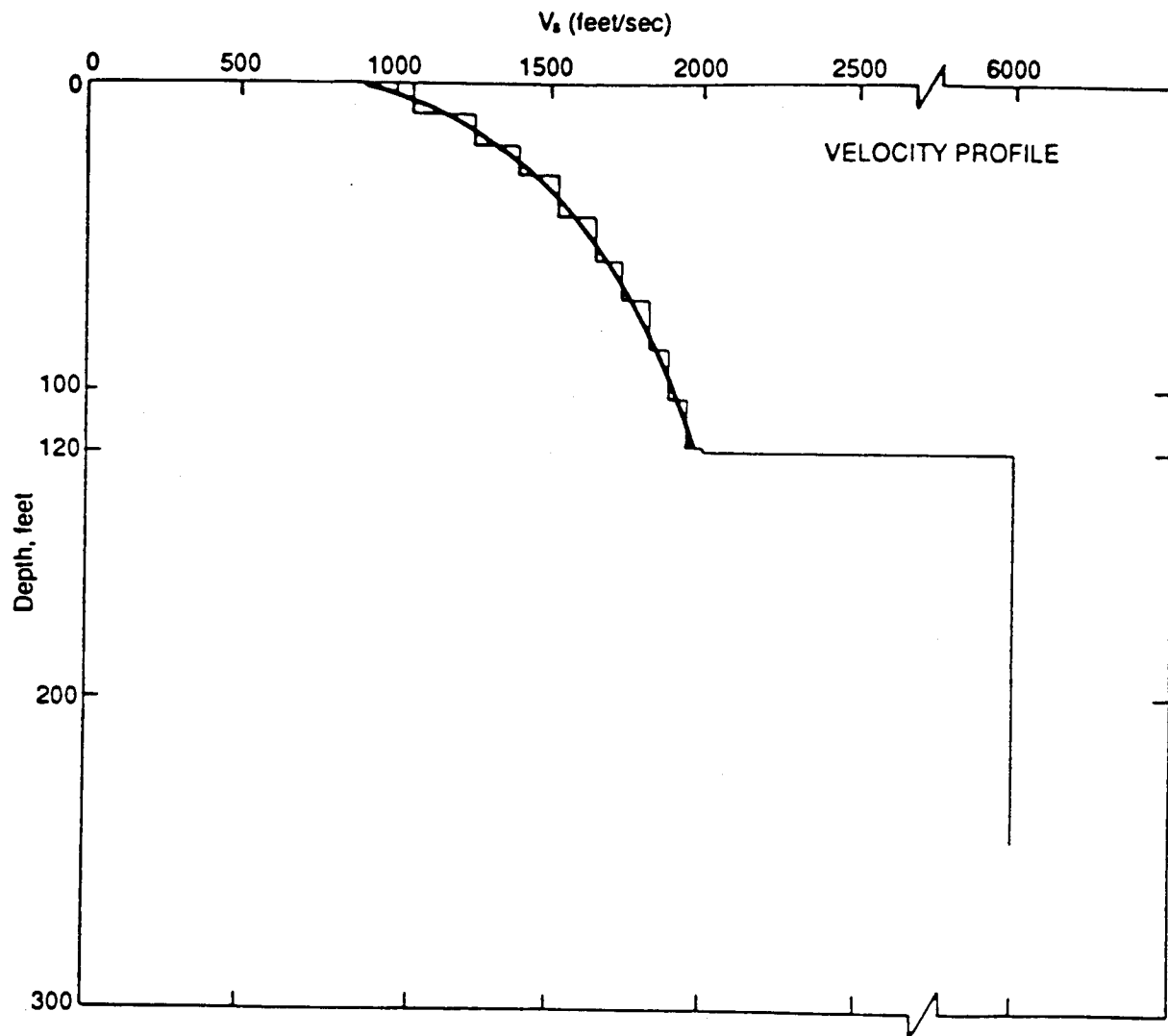


Figure 6.B-75
Generic soil profile used in the site response analyses with synthetic motions. Taken from Category 3 (Section 6).

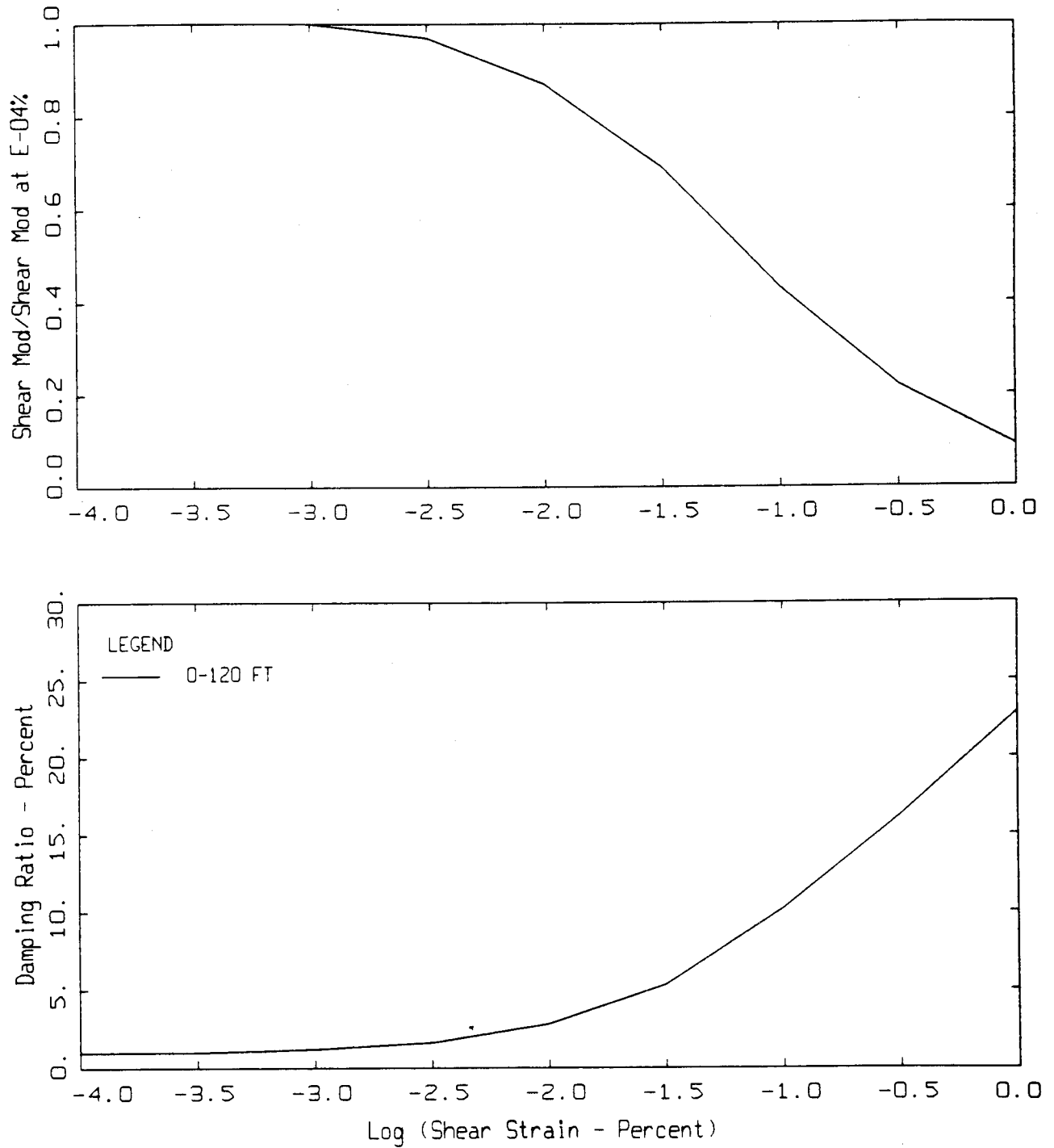


Figure 6.B-76
Modulus reduction and damping curves used in the site response analyses with synthetic motions for Site 3.

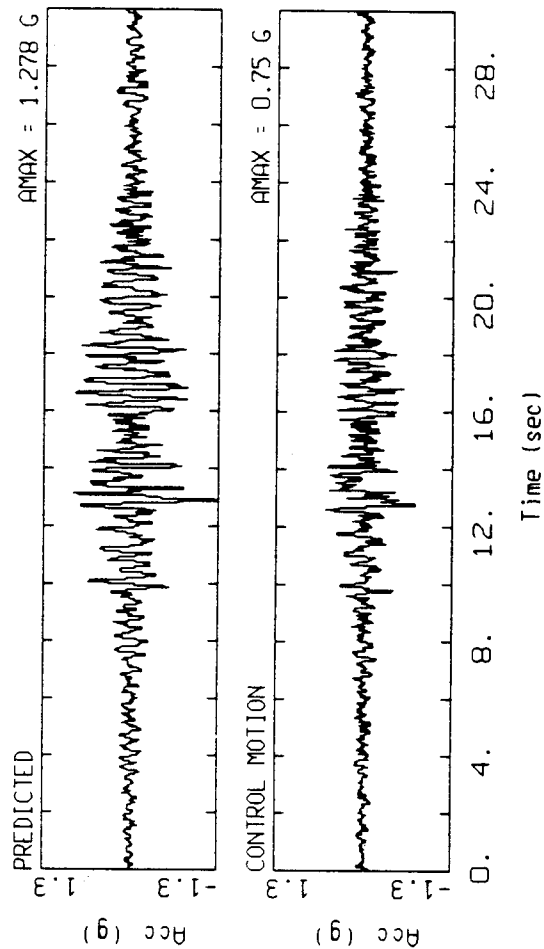
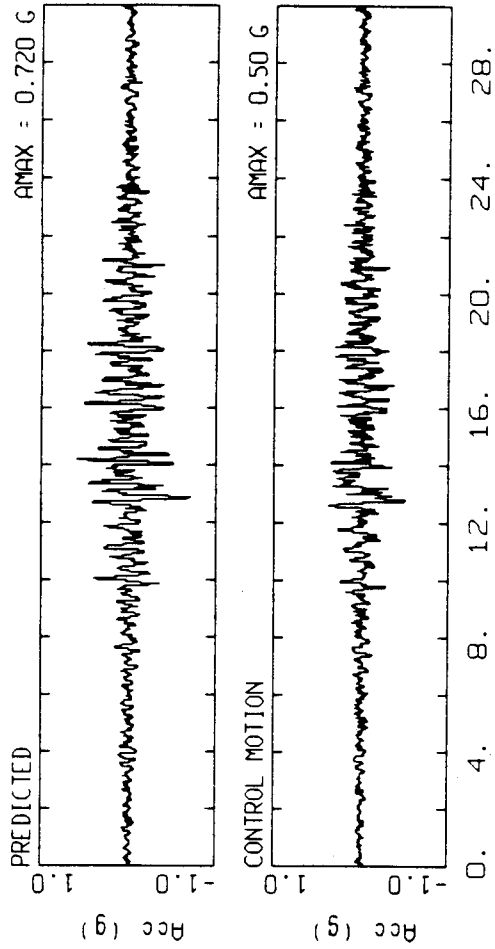
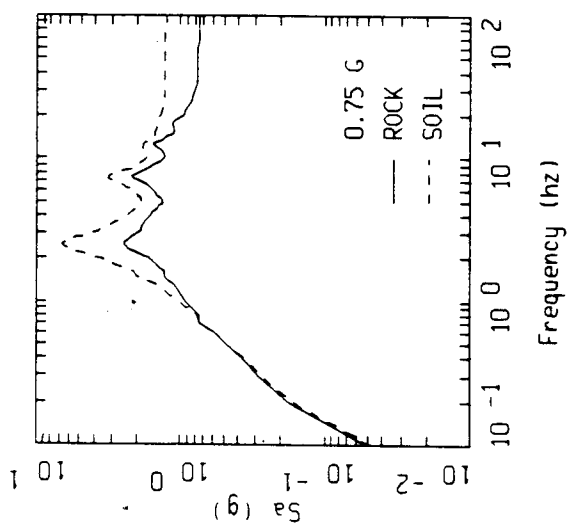
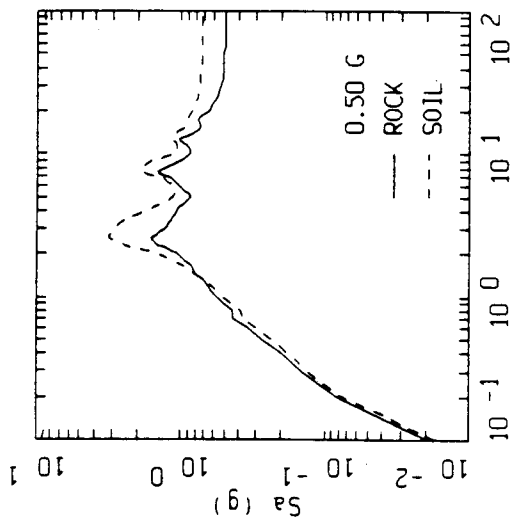


Figure 6.B-77

Predicted time histories and 5% damped spectral accelerations using the synthetic control motions of 0.50 and 0.75g. RASCAL equivalent-linear analysis.

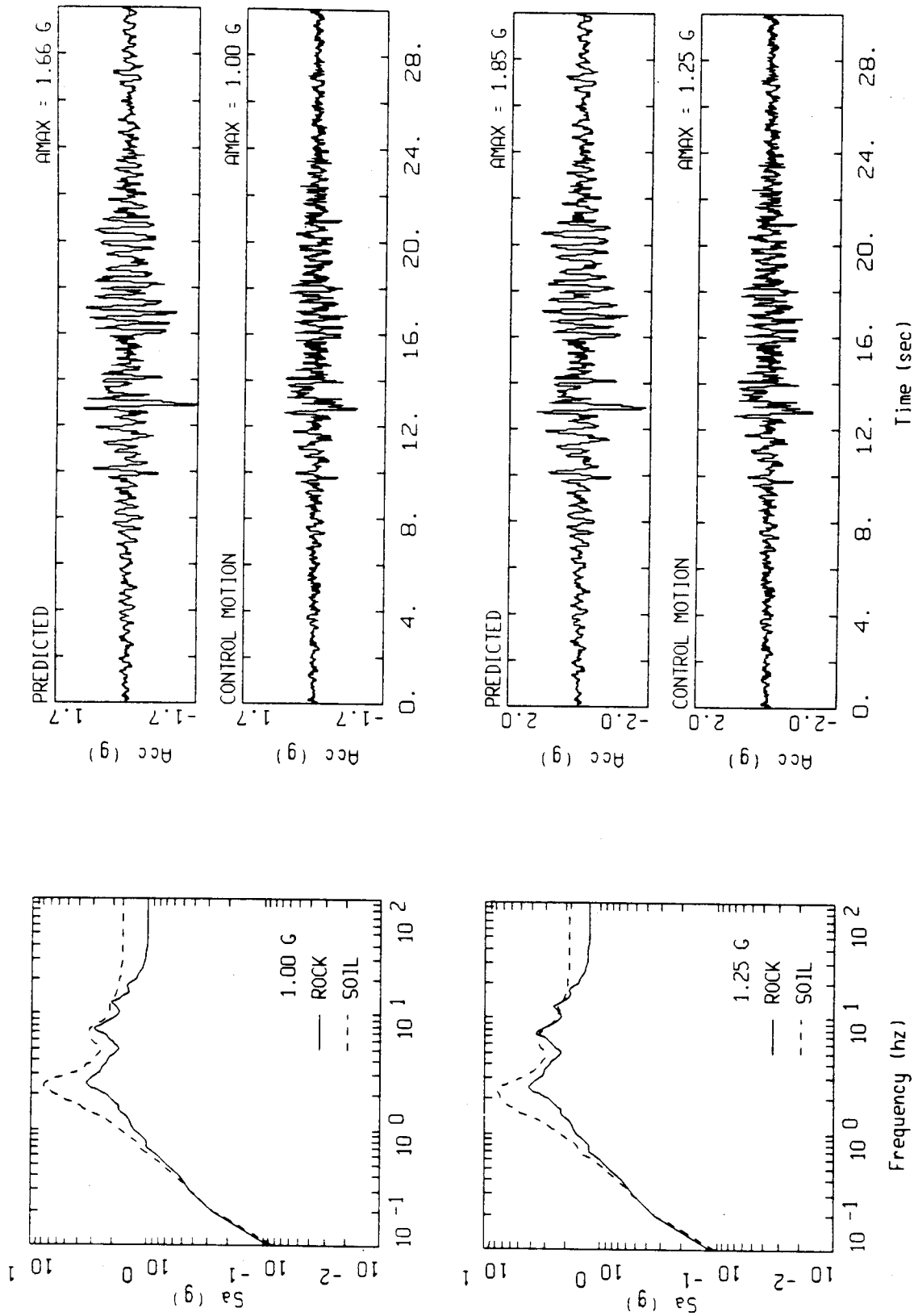


Figure 6.B-78

Predicted time histories and 5% damped spectral accelerations using the synthetic control motions of 1.00 and 1.25g. RASCAL equivalent-linear analysis.

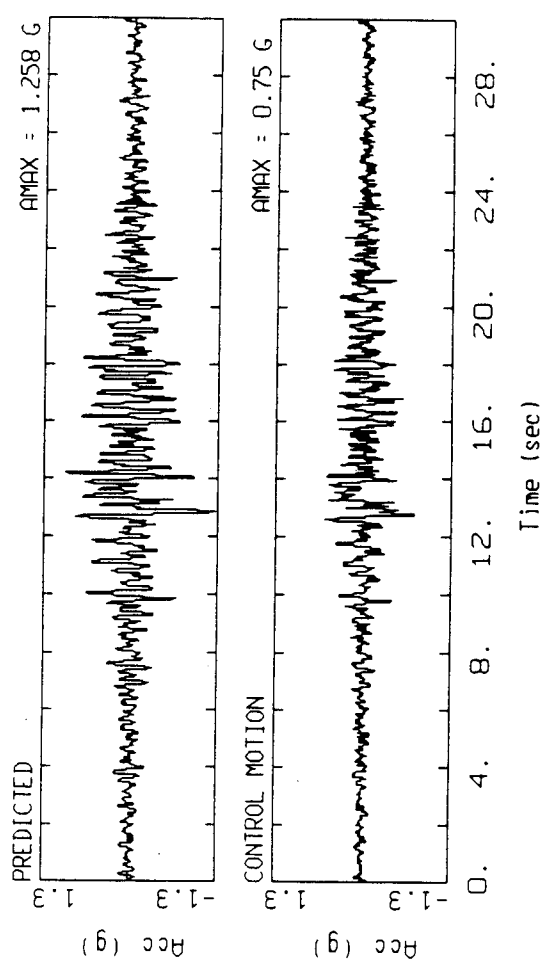
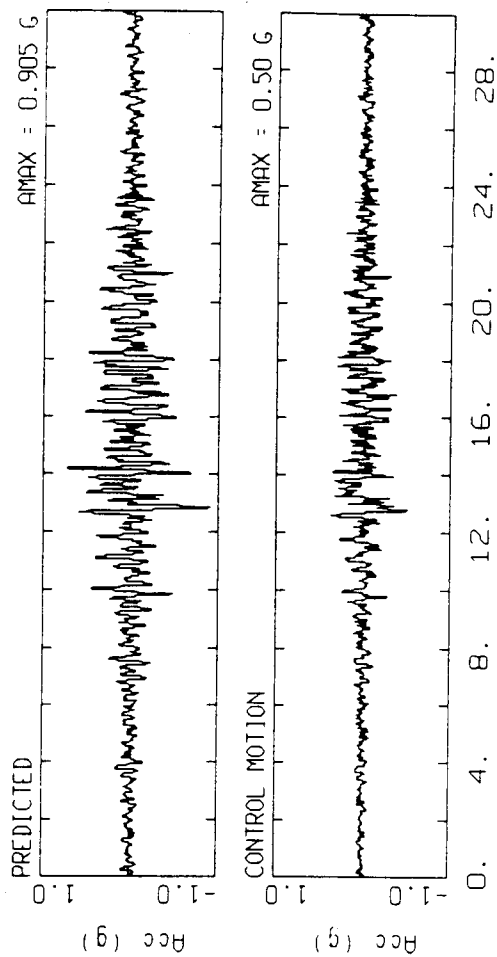
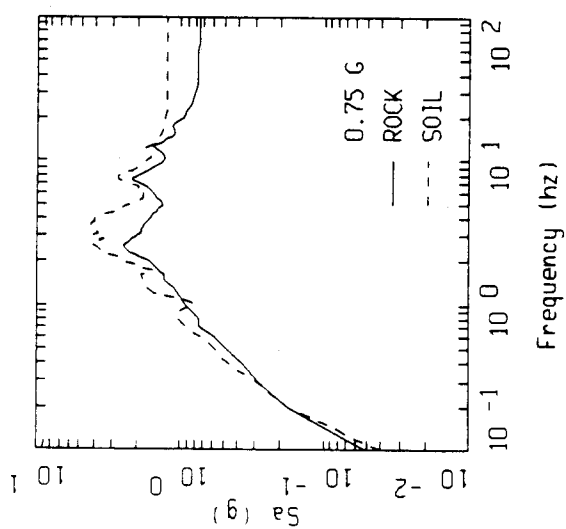
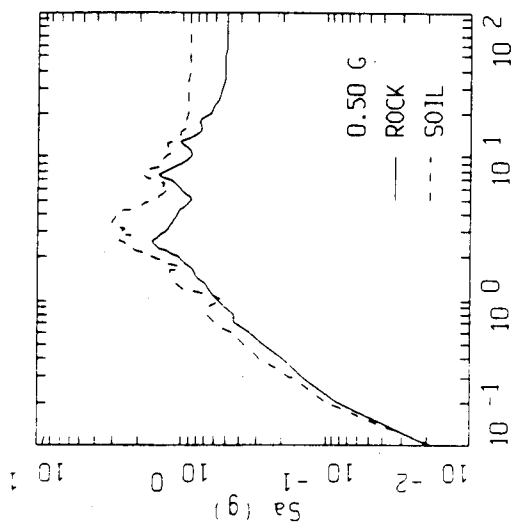


Figure 6.B-79

Predicted time histories and 5% damped spectral accelerations using the synthetic control motions of 0.50 and 0.75g. DESRA analysis.

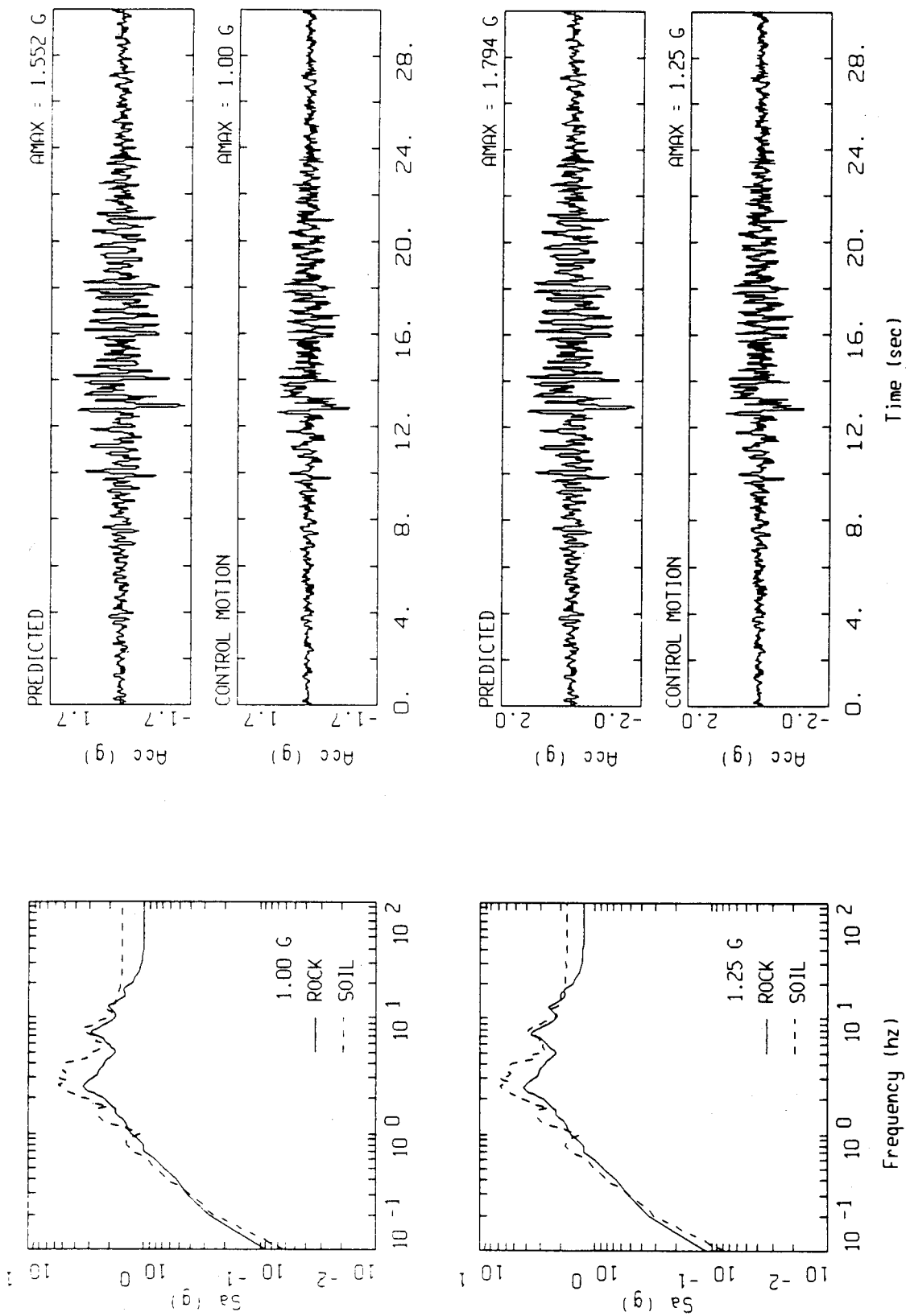


Figure 6.B-80
Predicted time histories and 5% damped spectral accelerations using the synthetic control motions of 1.00 and 1.25g. DESRA analysis.

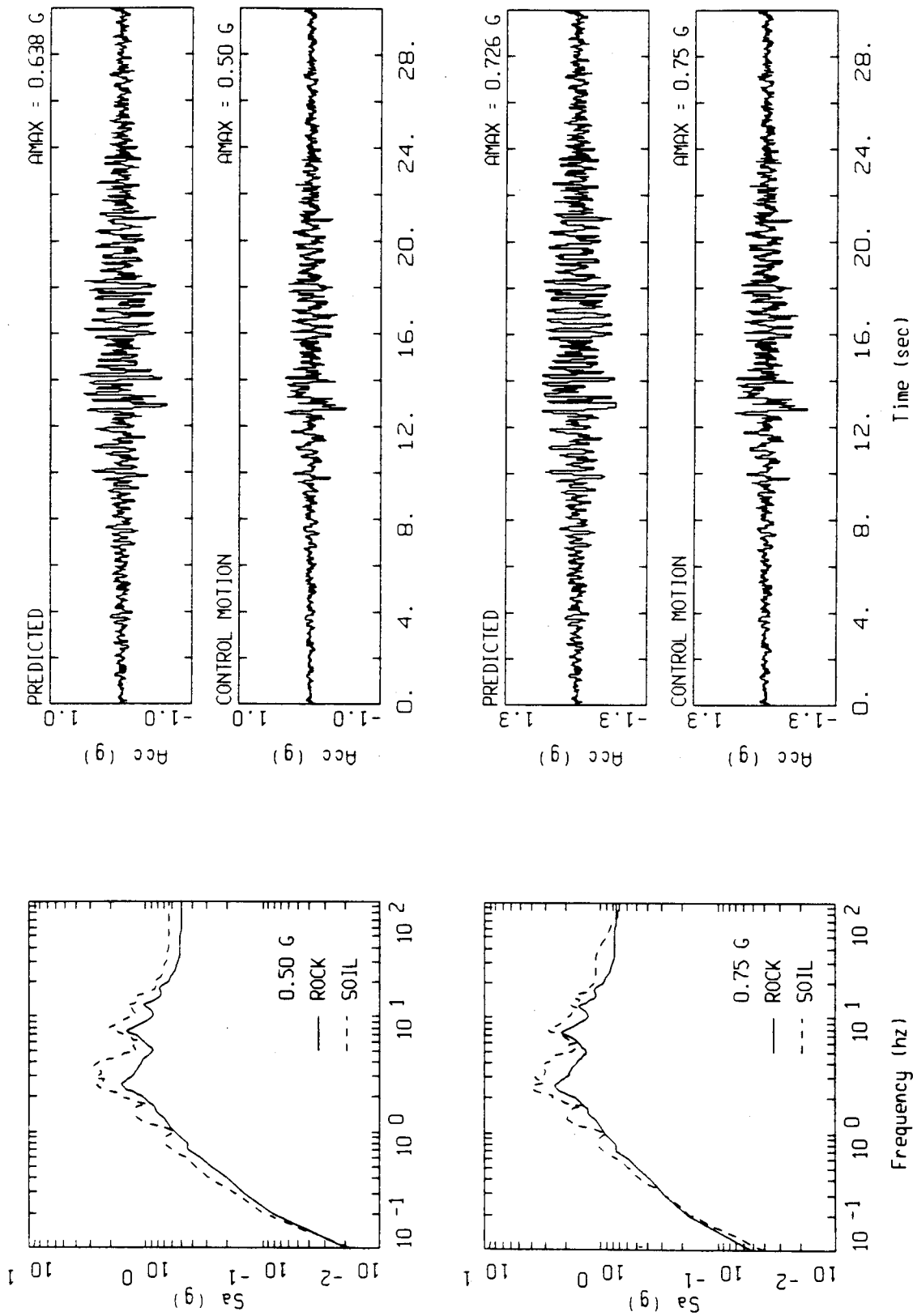


Figure 6.B-81
 Predicted time histories and 5% damped spectral accelerations using the synthetic control motions of 0.50 and 0.75g. SUMDES analysis.

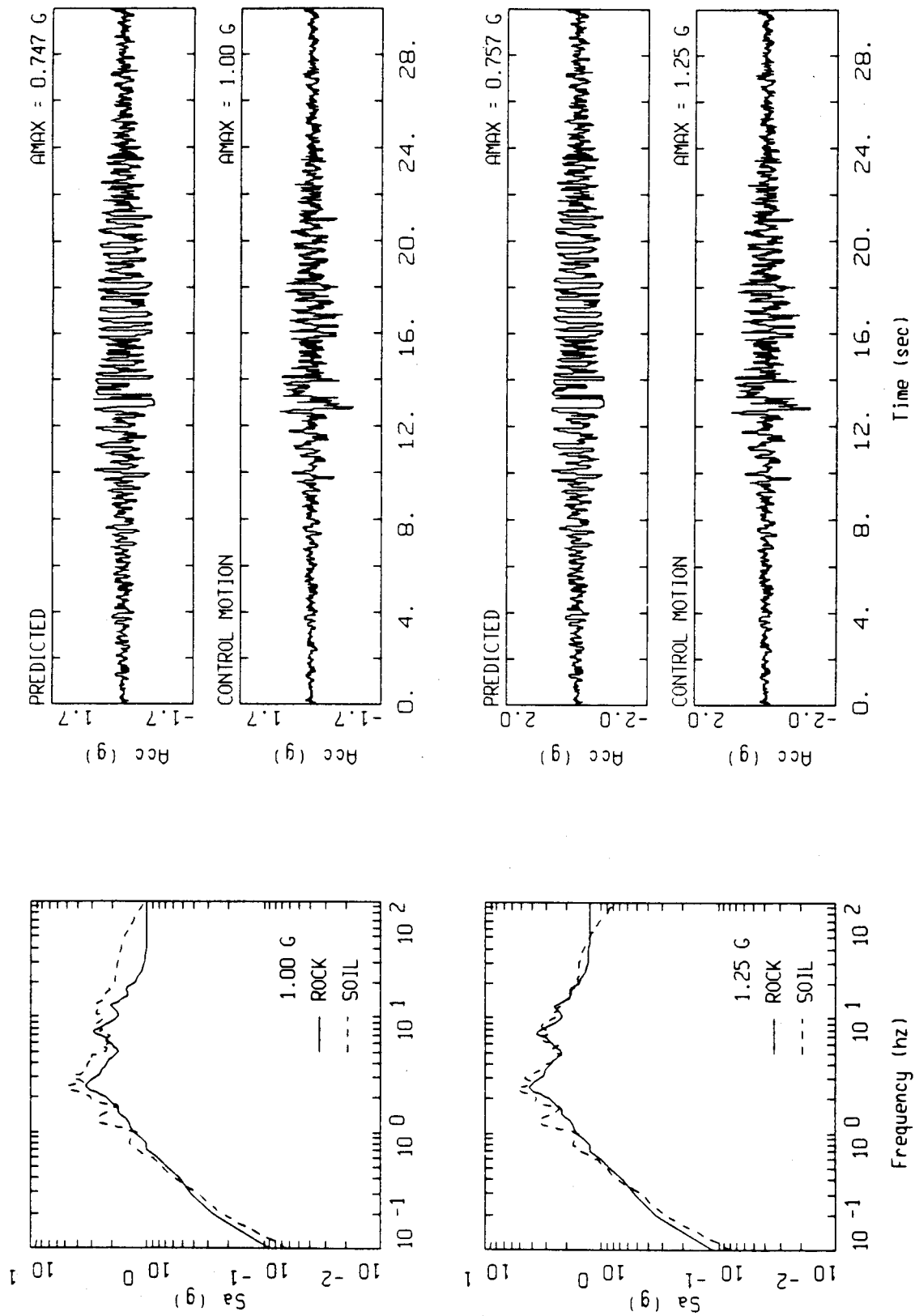


Figure 6.B-82
Predicted time histories and 5% damped spectral accelerations using the synthetic control motions of 1.00 and 1.25g. SUMDES analysis.

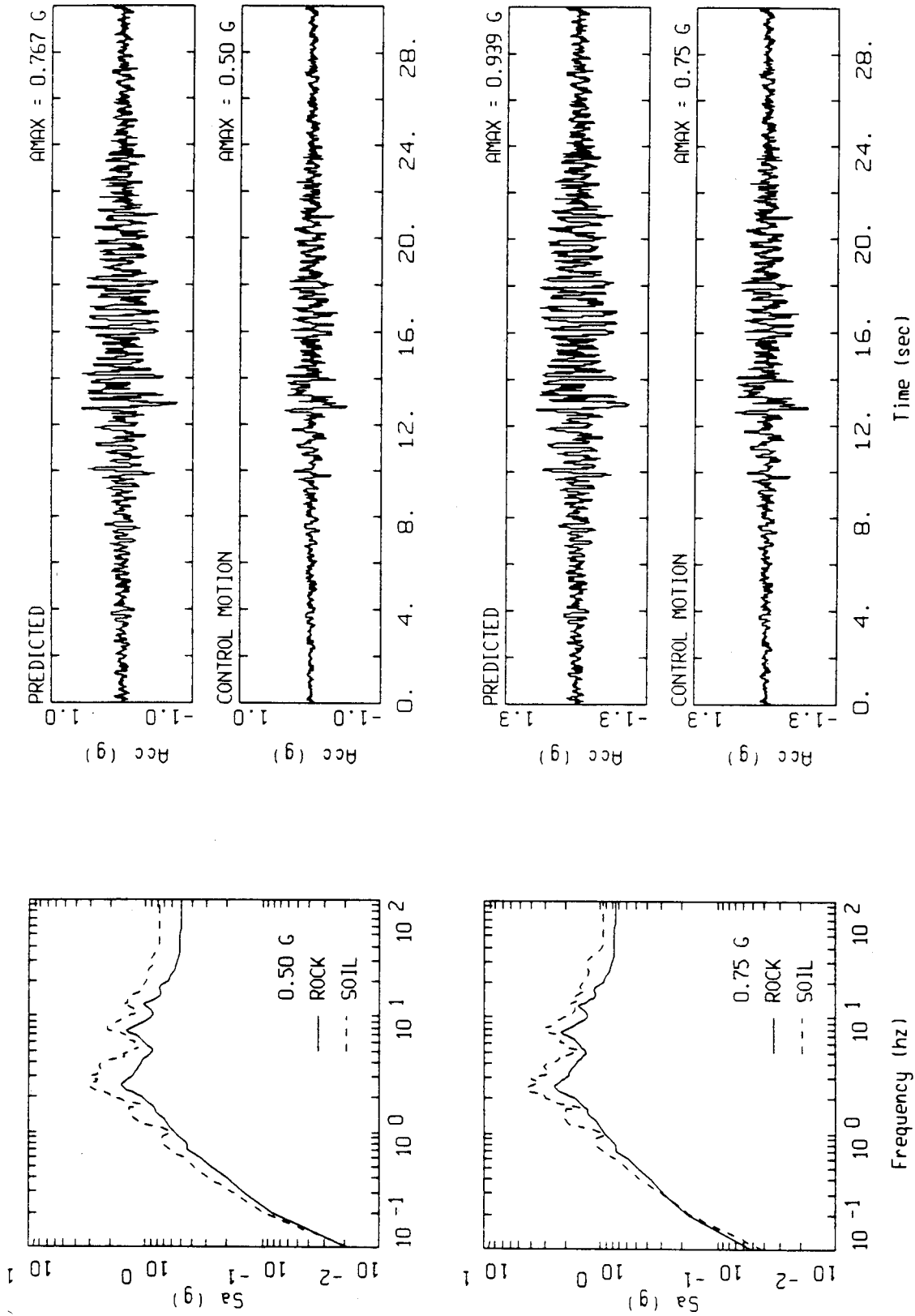


Figure 6.B-83
Predicted time histories and 5% damped spectral accelerations using the synthetic control motions of 0.50 and 0.75g. TESS analysis.

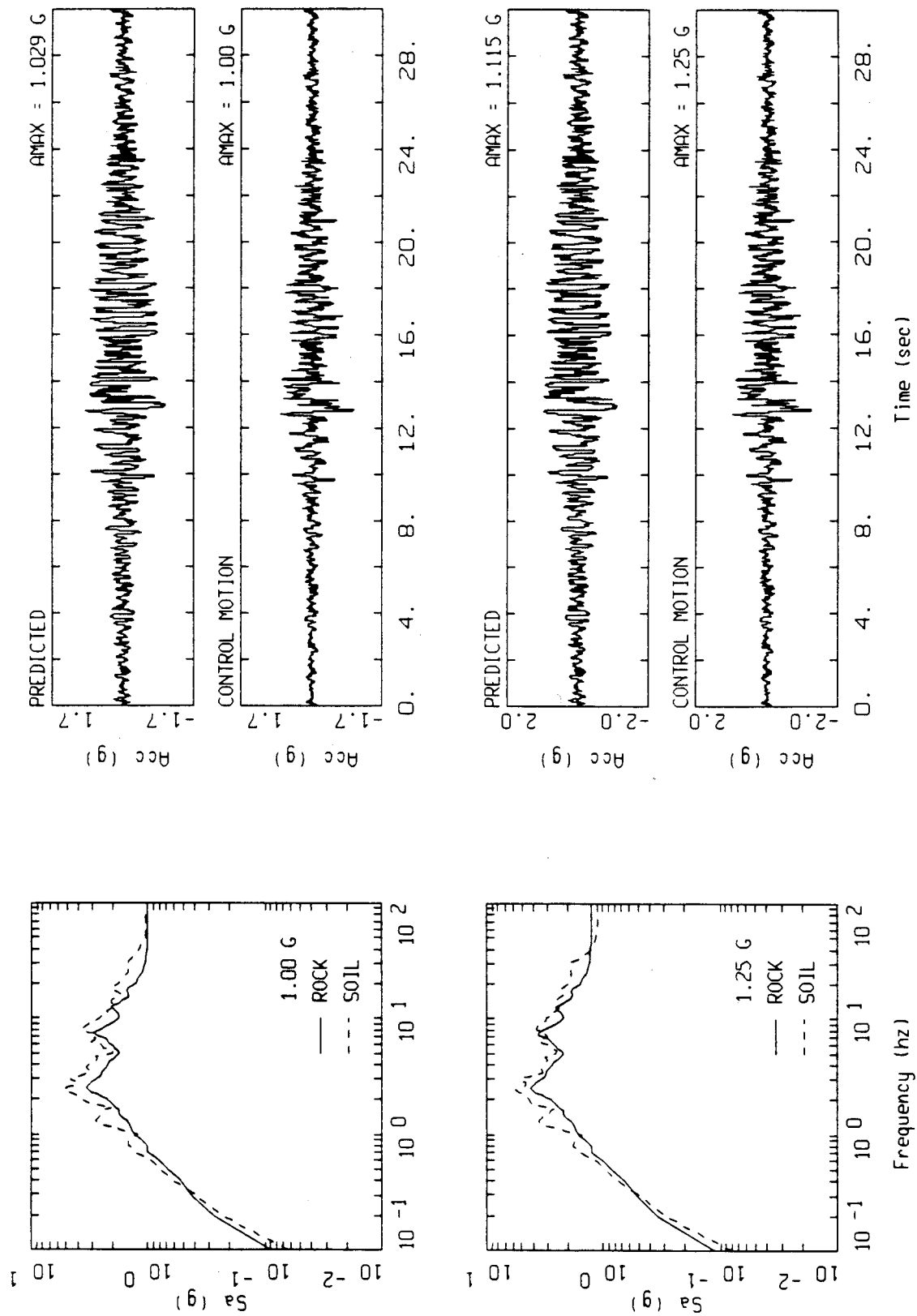


Figure 6.B-84
Predicted time histories and 5% damped spectral accelerations using the synthetic control motions of 1.00 and 1.25g TESS analysis.

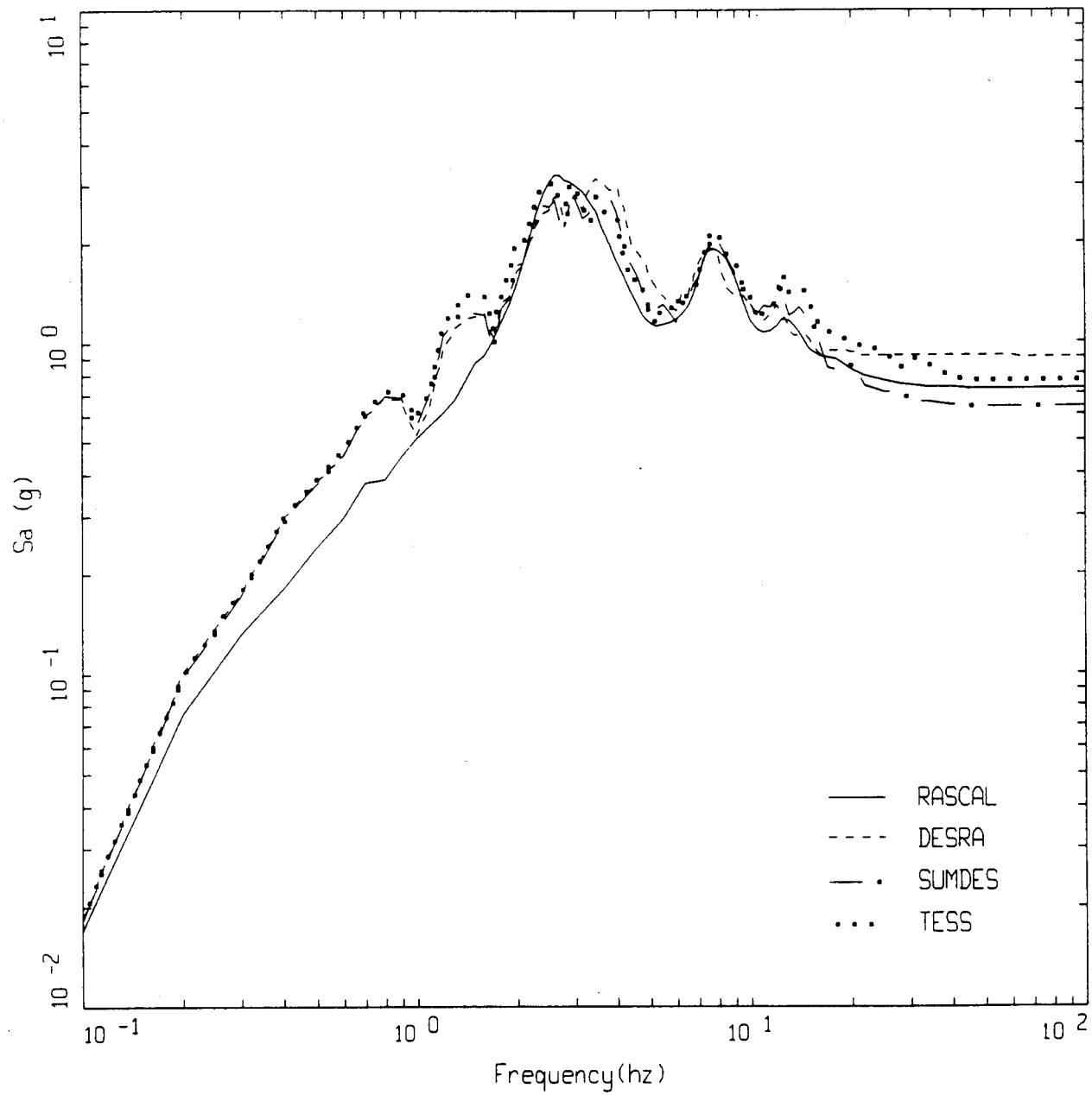


Figure 6.B-85

Comparison of 5% damped spectral accelerations for the four analyses using the synthetic control motions of 0.50g for Site 3.

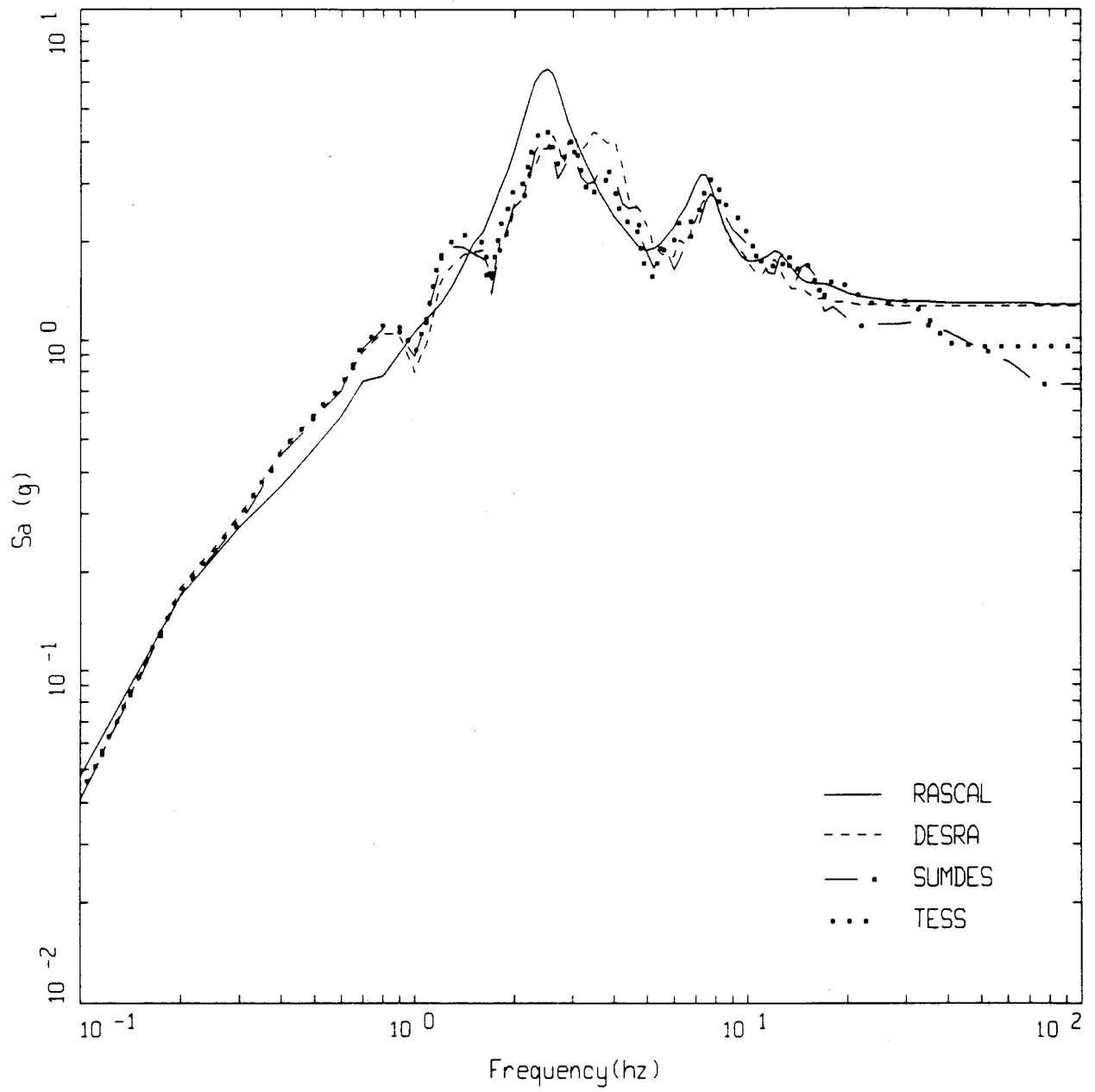


Figure 6.B-86

Comparison of 5% damped spectral accelerations for the four analyses using the synthetic control motions of 0.75g for Site 3.

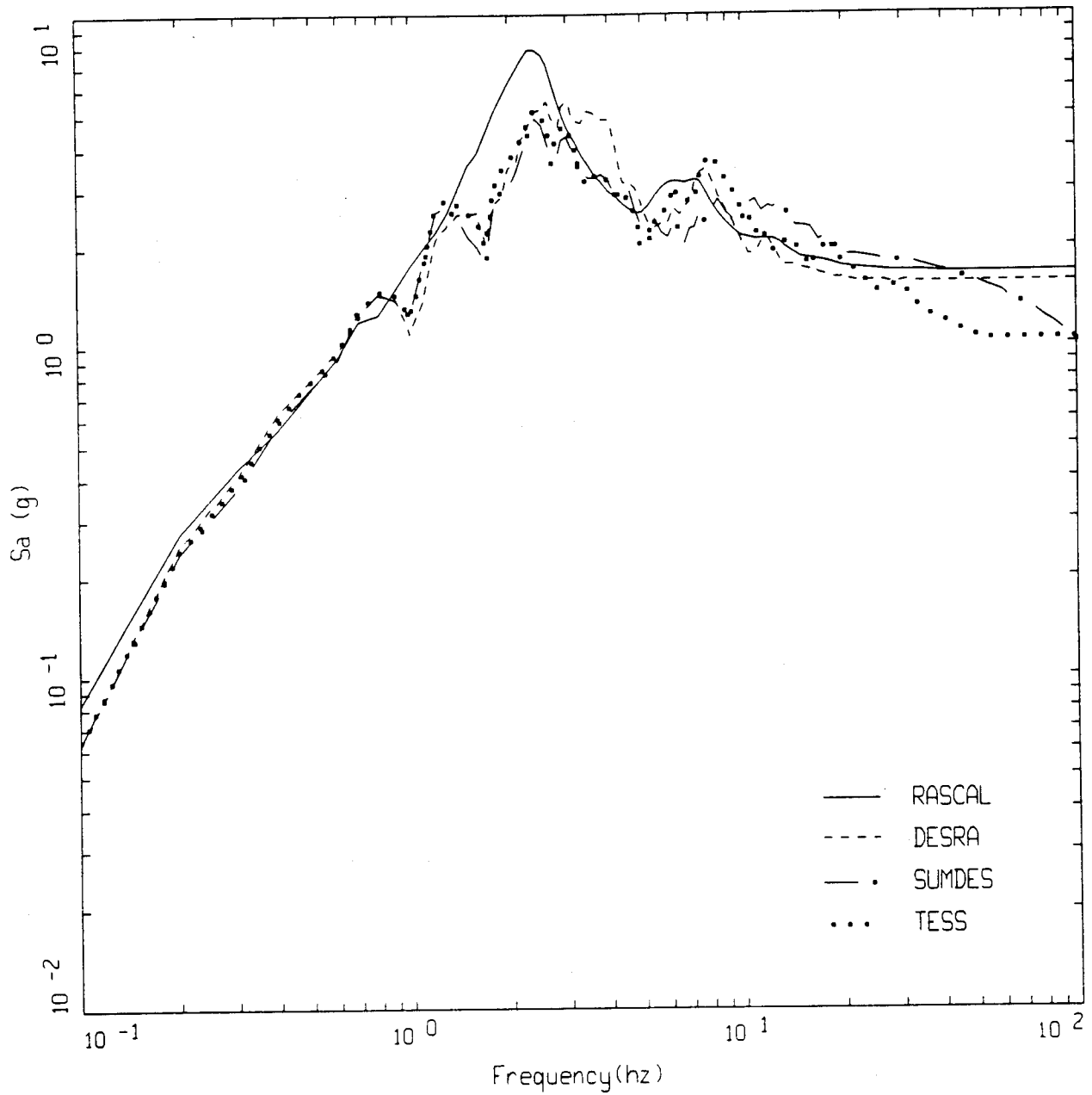


Figure 6.B-87

Comparison of 5% damped spectral accelerations for the four analyses using the synthetic control motions of 1.00g for Site 3.

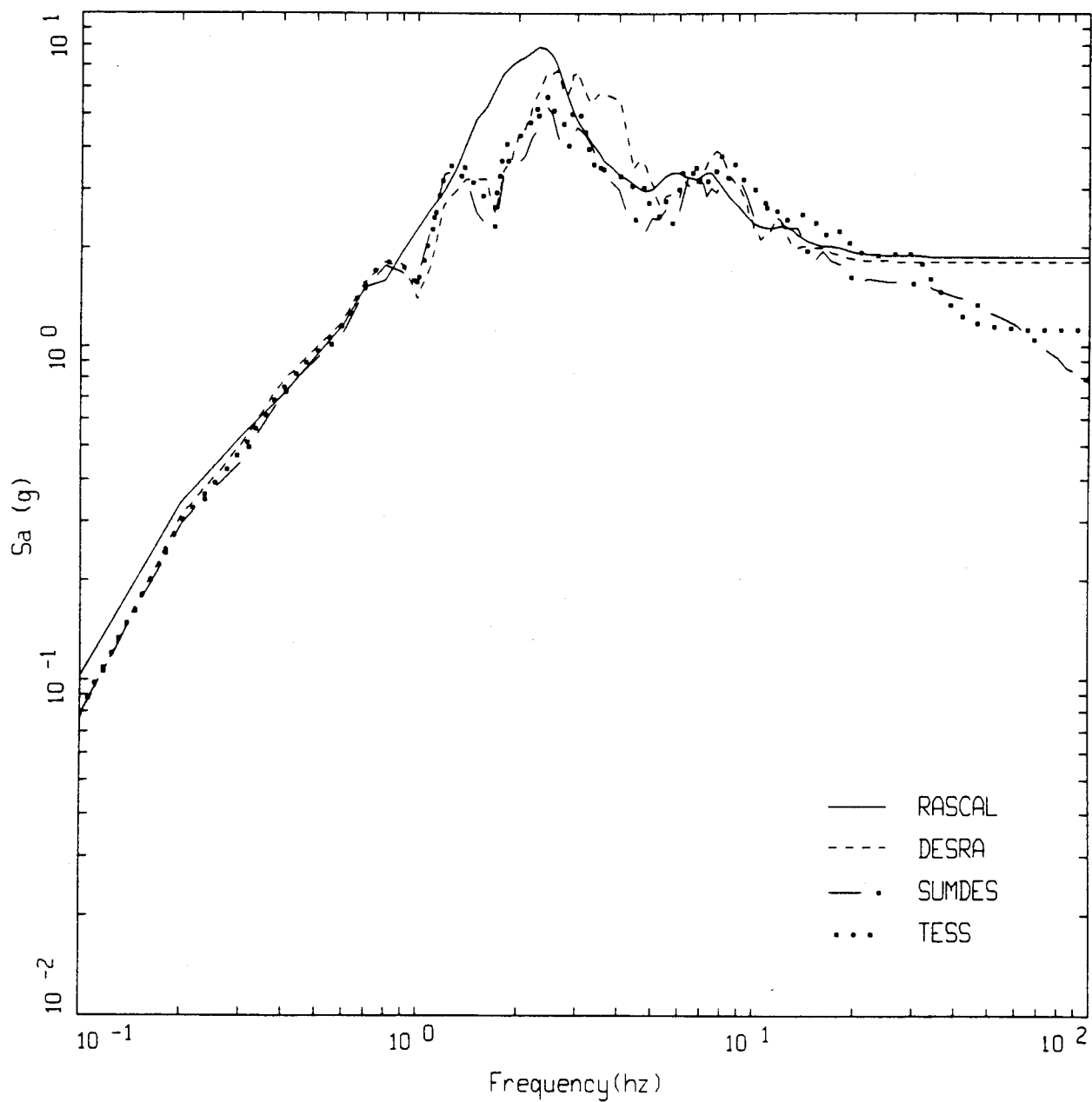


Figure 6.B-88

Comparison of 5% damped spectral accelerations for the four analyses using the synthetic control motions of 1.25g for Site 3.

Input Files for DESRA-2C, RASCAL, SUMDES, and TESS for the Gilroy 2 Analyses

Input File for DESRA-2,C, Site Gilroy 2

DESRA-2: Gilroy #2, Loma Prieta Earthquake, Gilroy #1 input

3	48	11	0	0	0	2	0	1	1	1	1
1	2		5.00			1.0	1				
1	2		5.00			1.0	1				
1	2		5.00			1.0	1				
1	2		6.67			1.0	1				
1	2		6.67			1.0	1				
1	2		6.66			1.0	1				
2	2		5.00			1.0	1				
3	2		5.00			1.0	1				
4	2		5.00			1.0	1				
4	2		5.00			1.0	1				
4	2		5.00			1.0	1				
4	2		5.00			1.0	1				
4	2		5.00			1.0	0				
4	2		5.00			1.0	0				
5	2		5.00			1.0	0				
5	2		5.00			1.0	0				
5	2		10.00			1.0	0				
5	2		10.00			1.0	0				
6	2		10.00			1.0	0				
6	2		10.00			1.0	0				
7	2		12.00			1.0	0				
8	2		12.00			1.0	0				
8	2		12.00			1.0	0				
8	2		12.00			1.0	0				
8	2		12.00			1.0	0				
8	2		12.00			1.0	0				
8	2		12.00			1.0	0				
8	2		12.00			1.0	0				
8	2		12.00			1.0	0				
8	2		12.00			1.0	0				
8	2		12.00			1.0	0				
9	2		11.00			1.0	0				
9	2		12.00			1.0	0				
10	2		14.00			1.0	0				
10	2		15.00			1.0	0				
11	2		19.00			1.0	0				
11	2		19.00			1.0	0				
11	2		20.00			1.0	0				
11	2		20.00			1.0	0				
11	2		20.00			1.0	0				

Input File for DESRA-2C, Site Gilroy 2 (continued)

11	2	20.00	1.0	0				
11	2	20.00	1.0	0				
11	2	20.00	1.0	0				
11	2	20.00	1.0	0				
11	2	20.00	1.0	0				
11	2	20.00	1.0	0				
11	2	20.00	1.0	0				
1550000.0	2168.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
118.0000	118.0000	0.0000	0.0000	0.0113				
3660000.0	5130.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
118.0000	118.0000	0.0000	0.0000	0.0113				
3660000.0	7686.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
118.0000	118.0000	0.0000	0.0000	0.0113				
9300000.0	19522.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
123.0000	123.0000	0.0000	0.0000	0.0113				
3760000.0	9394.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
121.0000	121.0000	0.0000	0.0000	0.0113				
4880000.0	12209.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
121.0000	121.0000	0.0000	0.0000	0.0113				
6110000.0	4276.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
130.0000	130.0000	0.0000	0.0000	0.0113				
17800000.0	12463.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
130.0000	130.0000	0.0000	0.0000	0.0113				
9700000.0	6790.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
130.0000	130.0000	0.0000	0.0000	0.0113				
12100000.0	8548.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
130.0000	130.0000	0.0000	0.0000	0.0113				
21360000.0	14950.0000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0025	0.4300	0.6200				
130.0000	130.0000	0.0000	0.0000	0.0113				
130.0	3900.0							
1	1600010001000	1	0.005					
1200	5	0	1.0000					

(5E15.7)

Input File for DESRA-2C, Site Gilroy 2 (continued)

Loma Prieta 10/17/89, at Gilroy #1, N-S component [g01000.A05]

.4908652E-02	.5422986E-02	.2539464E-02	-.1524564E-02	-.8657094E-03
-.8967781E-03	-.9692611E-03	-.1062645E-02	-.1132162E-02	-.1157966E-02
-.1177797E-02	-.1218719E-02	-.1256826E-02	-.1275379E-02	-.1302567E-02
-.1354722E-02	-.1408868E-02	-.1448177E-02	-.1473374E-02	-.1485829E-02

(remainder of time history omitted)

Input Files for RASCAL, Site Gilroy 2

A)

LOMA PRIETA AT GILROY 2, NS COMP, BASE-CASE PROFILE

NS

C:\freq\nrcL.frq

200,2.7,1.0,00.0,3.2,0.0

10000000.0,1.0,0.6,0.00,7.0,6.4,1.0,0.,0.,1.

5.0,0,0,0.000

2,0,0,0,-1,0,1,1,0

BASE12.PAR

..\..\MATERIAL\EPRIG25.MAT

..\..\DATA\G01000.A05

7990 76 0.005 (5E15.7)

-1. -1. -1.

0 0

62.5 0 0.10 -0

..\..\WESSA\G01000.FAS

2049 95(2E15.7)

B)

GILROY 2 FINAL BASE PROFILE, EPRI CURVES

1 37 37 7 0 0 3 1 37 0

.65 1.00 1.00 1.00 .05 1.00 0. 0.

.00 .00 .00

2.133 198.11 1.930 3.00 2

2.133 198.11 1.930 3.00 2

2.133 198.11 1.930 3.00 2

2.134 198.11 1.930 3.00 2

2.134 198.11 1.930 3.00 2

3.048 304.79 1.930 3.00 2

4.573 475.47 1.930 3.00 2

4.572 475.47 1.930 3.00 2

3.149 304.79 1.930 3.00 2

3.149 304.79 1.930 3.00 2

3.149 304.79 1.930 3.00 2

3.048 347.46 2.130 3.00 2

3.048 347.46 2.130 3.00 2

3.657 374.89 2.130 3.00 3

5.747 640.05 2.130 3.00 3

5.747 640.05 2.130 3.00 3

Input Files for RASCAL, Site Gilroy 2 (continued)

B)

5.747	640.05	2.130	3.00	3
5.747	640.05	2.130	3.00	3
5.747	640.05	2.130	3.00	3
5.747	640.05	2.130	3.00	3
5.748	640.05	2.130	3.00	3
3.505	472.42	2.130	3.00	3
3.505	472.42	2.130	3.00	3
4.419	527.28	2.130	3.00	3
4.420	527.28	2.130	3.00	3
4.723	701.01	2.130	3.00	3
6.781	701.01	2.130	3.00	3
6.781	701.01	2.130	3.00	3
6.782	701.01	2.130	3.00	3
6.782	701.01	2.130	3.00	3
6.782	701.01	2.130	3.00	3
6.782	701.01	2.130	3.00	3
6.782	701.01	2.130	3.00	3
6.782	701.01	2.130	3.00	3
6.782	701.01	2.130	3.00	3
6.782	701.01	2.130	3.00	3
0.00	1188.66	2.600	1.667	-1

C)

1. UNITY.MOD: CONSTANT MODULUS CURVE

1.0	1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.00	1.00						

1. UNITY DAMPING CURVE

1.0	1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.00	1.00						

3. EPRI SITING MOD RED FOR GILROY2, 00-130 FT (00-39.62 M)

1.0	1.0	1.00	0.99	0.93	0.76	0.51	0.27
0.11	0.06						

3. EPRI SITING DAMPING FOR GILROY2, 10-130 FT (00-39.62 M)

3.0	3.0	3.0	3.3	4.40	6.90	11.2	17.3
23.5	26.5						

5. EPRI SITING MOD RED FOR GILROY2, 130+ FT (39.62+ M)

1.0	1.0	1.00	0.95	0.81	0.57	0.32	0.14
0.05	0.02						

5. EPRI SITING DAMPING FOR GILROY2, 130+ FT (39.62+ M)

3.0	3.0	3.3	4.1	6.10	10.0	15.9	22.2
27.5	30.2						

Input File for SUMDES, Site Gilroy 2

[Title of analysis]

SUMDES ver. 4.0: Gilroy #2 test case

[Global information: The basic units used in the file are m, kg, and sec]

101300.0 9.81 9810.0

2300.0 1188.7 3200.0

41 22 22

(Elastic base, input motion from Gilroy #1 record of Loma Prieta EQ)

[Layer properties:]

0.7	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	1
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	1
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	1
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	1
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	1
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	1
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	1
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	1
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	1
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	1
1.05	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	2
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	2
1.0	2.7	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	2
1.0	2.65	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	3
1.0	2.65	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	3
1.0	2.65	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	3
1.0	2.65	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	3
1.0	2.65	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	3
1.0	2.65	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	3
1.0	2.65	18933.0	0.0E0	0.75	0.005	0.005	0.0E0	3
1.0	2.65	18933.0	1.0E-5	0.75	0.005	0.005	8.2E8	3
1.4	2.65	18933.0	1.0E-5	0.75	0.005	0.005	8.2E8	3
4.75	2.7	18933.0	1.0E-8	0.75	0.005	0.005	4.9E9	4
4.75	2.7	18933.0	1.0E-8	0.75	0.005	0.005	4.9E9	4
6.1	2.65	20895.0	1.0E-8	0.75	0.005	0.005	5.3E9	5
3.7	2.65	20895.0	1.0E-5	0.75	0.005	0.005	5.9E9	6
4.1	2.65	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	7
10.0	2.65	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	7
10.0	2.65	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	7
10.0	2.65	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	7
6.1	2.65	20895.0	1.0E-5	0.75	0.005	0.005	6.0E9	8

Input File for SUMDES, Site Gilroy 2 (continued)

7.0	2.65	20895.0	1.0E-5	0.75	0.005	0.005	4.7E9	9
8.8	2.65	20895.0	1.0E-5	0.75	0.005	0.005	6.3E9	10
12.5	2.7	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	11
10.0	2.7	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	11
10.0	2.7	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	11
10.0	2.7	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	11
10.0	2.7	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	11
10.0	2.7	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	11
10.0	2.7	20895.0	1.0E-5	0.75	0.005	0.005	7.8E9	11

(The coefficient of permeability and the bulk modulus of pore fluid for the top layers are dummy values.)

[Model definition:]

1	4	30.0	326.0	0.0051	0.0010	2.072	100.0	0.75	100.0	2.0	35.0	/
2	4	30.0	216.0	0.0078	0.0015	1.068	100.0	0.75	100.0	2.0	35.0	/
3	4	30.0	930.0	0.0016	0.0003	3.994	100.0	0.75	100.0	2.0	35.0	/
4	4	30.0	310.0	0.0054	0.0011	0.979	100.0	0.75	100.0	2.0	35.0	/
5	4	30.0	238.0	0.0033	0.0006	0.761	100.0	0.75	100.0	2.0	35.0	/
6	4	30.0	259.0	0.0030	0.0006	0.876	100.0	0.75	100.0	2.0	35.0	/
7	4	30.0	624.0	0.0002	0.0012	0.546	100.0	0.75	100.0	2.0	35.0	/
8	4	30.0	572.0	0.0003	0.0015	0.434	100.0	0.75	100.0	2.0	35.0	/
9	4	30.0	300.0	0.0028	0.0006	0.191	100.0	0.75	100.0	2.0	35.0	/
10	4	30.0	357.0	0.0024	0.0005	0.235	100.0	0.75	100.0	2.0	35.0	/
11	4	30.0	528.0	0.0016	0.0003	0.387	100.0	0.75	100.0	2.0	35.0	/

(Model ID No. 1 - 11: Use the reduced-order bounding surface model Model 4)

[Input acceleration:]

```

0.0  0.005  4000
1.0  1.0    1.0
..\..\data\G01000.A05
76
(5E15.7)
DUMMY
0
Dummy format
DUMMY
0
Dummy format

```

(Only horizontal direction 1 is active)

Input File for SUMDES, Site Gilroy 2 (continued)

[Numerical control parameters]

0.0 0.25 0.5
0.001 5 16 0.1 5.0

[Output files]

g2-3ns.out
4 0.50
g2-3ns.sum

Input File for TESS, Site Gilroy 2

GILROY 2, INPUT LOMA PRIETA AT GILROY 1, NORTH-SOUTH
BASE CASE PROFILE

C NLAYERS NUMMAT KDIS NRUN
31 1 0 1

C PATM DEPTH SLOPE
2.116 70. 0.

C NLAYOUT DTIME
1 0.

C MATERIAL PROPERTIES
C MTYPE KGAT KDEG KCON VG VT
1 -2 0 0 0.06 0.06

C LAYER DATA

C No. Matype Thick. Unit Wt.
1 1 10. .1204
C Vs GAMr
650. .12

C No. Matype Thick. Unit Wt.
2 1 5. .1204
C Vs GAMr
650. .12

C No. Matype Thick. Unit Wt.
3 1 5. .1204
C Vs GAMr
650. .12

C No. Matype Thick. Unit Wt.
4 1 5. .1204
C Vs GAMr
650. .12

C No. Matype Thick. Unit Wt.
5 1 5. .1204
C Vs GAMr
650. .12

Input File for TESS, Site Gilroy 2 (continued)

C No. Matype Thick. Unit Wt.
6 1 5. .1204
C Vs GAMr
650. .12

C No. Matype Thick. Unit Wt.
7 1 10. .1204
C Vs GAMr
1000. .12

C No. Matype Thick. Unit Wt.
8 1 10. .1204
C Vs GAMr
1560. .12

C No. Matype Thick. Unit Wt.
9 1 10. .1204
C Vs GAMr
1560. .12

C No. Matype Thick. Unit Wt.
10 1 10. .1204
C Vs GAMr
1560. .12

C No. Matype Thick. Unit Wt.
11 1 10. .1204
C Vs GAMr
1000. .12

C No. Matype Thick. Unit Wt.
12 1 10. .1204
C Vs GAMr
1000. .12

C No. Matype Thick. Unit Wt.
13 1 11. .1204
C Vs GAMr
1000. .12

Input File for TESS, Site Gilroy 2 (continued)

C No.	Matype	Thick.	Unit Wt.
14 1	10.	.1329	
C Vs	GAMr		
1140.	.12		

C No.	Matype	Thick.	Unit Wt.
15 1	10.	.1329	
C Vs	GAMr		
1140.	.12		

C No.	Matype	Thick.	Unit Wt.
16 1	12.	.1329	
C Vs	GAMr		
1230.	.05		

C No.	Matype	Thick.	Unit Wt.
17 1	12.	.1329	
C Vs	GAMr		
2100.	.05		

C No.	Matype	Thick.	Unit Wt.
18 1	20.	.1329	
C Vs	GAMr		
2100.	.05		

C No.	Matype	Thick.	Unit Wt.
19 1	20.	.1329	
C Vs	GAMr		
2100.	.05		

C No.	Matype	Thick.	Unit Wt.
20 1	20.	.1329	
C Vs	GAMr		
2100.	.05		

C No.	Matype	Thick.	Unit Wt.
21 1	20.	.1329	
C Vs	GAMr		
2100.	.05		

Input File for TESS, Site Gilroy 2 (continued)

C No.	Matype	Thick.	Unit Wt.
22	1	20.	.1329
C Vs	GAMr		
2100.	.05		

C No.	Matype	Thick.	Unit Wt.
23	1	20.	.1329
C Vs	GAMr		
2100.	.05		

C No.	Matype	Thick.	Unit Wt.
24	1	23.	.1329
C Vs	GAMr		
1550.	.05		

C No.	Matype	Thick.	Unit Wt.
25	1	29.	.1329
C Vs	GAMr		
1730.	.05		

C No.	Matype	Thick.	Unit Wt.
26	1	38.	.1329
C Vs	GAMr		
2300.	.05		

C No.	Matype	Thick.	Unit Wt.
27	1	40.	.1329
C Vs	GAMr		
2300.	.05		

C No.	Matype	Thick.	Unit Wt.
28	1	40.	.1329
C Vs	GAMr		
2300.	.05		

C No.	Matype	Thick.	Unit Wt.
29	1	40.	.1329
C Vs	GAMr		
2300.	.05		

Input File for TESS, Site Gilroy 2 (continued)

C No. Matype Thick. Unit Wt.
30 1 40. .1329
C Vs GAMr
2300. .05

C No. Matype Thick. Unit Wt.
31 1 40. .1329
C Vs GAMr
2300. .05

C COMPLIANT BASE DATA

C Vs Unit Wt. Kode
3900 0.1435 0

C OUTPUT OPTIONS

C To save surface acceleration history

C Layer No. No. of histories
1 3

C Codes for type of histories
1 4 5

C INPUT MOTION

GILROY 1, LOMA PRIETA, NORTH-SOUTH

C Code Code NPTS NTOT DT Amax KREAD
1 0 7990 7990 0.005 0.4108 5

.4908652E-02 .5422986E-02 .2539464E-02 -.1524564E-02 -.8657094E-03
-.8967781E-03 -.9692611E-03 -.1062645E-02 -.1132162E-02 -.1157966E-02
-.1177797E-02 -.1218719E-02 -.1256826E-02 -.1275379E-02 -.1302567E-02
-.1354722E-02 -.1408868E-02 -.1448177E-02 -.1473374E-02 -.1485829E-02

(remainder of time history omitted)

Input Files for DESRA-2C, RASCAL, SUMDES, and TESS for the Treasure Island Analyses

Input Files for DESRA-2C, Site Treasure Island

DESRA-2: Treasure Island site, Loma Prieta Earthquake, Yerba Buena Is. input

3	46	12	0	0	0	2	0	1	1	1	1
1	2		1.64			1.0	1				
1	2		3.28			1.0	1				
1	2		3.28			1.0	0				
2	2		3.28			1.0	0				
2	2		3.28			1.0	0				
2	2		3.28			1.0	0				
2	2		3.28			1.0	0				
2	2		4.92			1.0	0				
3	2		3.28			1.0	0				
3	2		3.28			1.0	0				
3	2		3.28			1.0	0				
3	2		3.28			1.0	0				
3	2		3.61			1.0	0				
4	2		3.28			1.0	0				
4	2		3.28			1.0	0				
4	2		3.28			1.0	0				
4	2		3.28			1.0	0				
4	2		3.94			1.0	0				
5	2		3.28			1.0	0				
5	2		3.28			1.0	0				
5	2		3.28			1.0	0				
6	2		3.28			1.0	0				
6	2		3.28			1.0	0				
6	2		3.28			1.0	0				
6	2		3.28			1.0	0				
6	2		3.28			1.0	0				
6	2		3.28			1.0	0				
6	2		4.92			1.0	0				
7	2		7.71			1.0	0				
7	2		7.71			1.0	0				
8	2		10.00			1.0	0				
8	2		10.00			1.0	0				
8	2		10.00			1.0	0				
8	2		10.00			1.0	0				
9	2		13.95			1.0	0				
9	2		13.95			1.0	0				
9	2		13.95			1.0	0				
9	2		13.95			1.0	0				
10	2		10.02			1.0	0				
10	2		10.02			1.0	0				
10	2		10.02			1.0	0				

Input Files for DESRA-2C, Site Treasure Island (continued)

10	2	10.02	1.0	0				
11	2	21.32	1.0	0				
11	2	21.32	1.0	0				
12	2	19.68	1.0	0				
12	2	19.68	1.0	0				
1231600.0	998.8000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0018	0.4300	0.6200				
111.4000	120.0000	0.0000	0.0000	0.0025				
714700.0	579.6000	0.0000	0.0000	0.0000	0.00000			
0.5000	0.8000	0.0018	0.4300	0.6200				
111.4000	120.0000	0.0005	0.0000	0.0025				
1272700.0	1032.1000	0.0000	0.0000	0.0000	0.00000			
0.5000	0.8000	0.0018	0.4300	0.6200				
120.0000	120.0000	0.0005	0.0000	0.0025				
1169300.0	3507.9000	0.0000	0.0000	0.0000	0.00000			
0.7500	0.5300	0.0015	0.4300	0.6200				
108.0000	108.0000	0.0005	0.0000	0.0025				
1587800.0	4763.4000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0015	0.4300	0.6200				
108.0000	108.0000	0.00000001	0.0000	0.0025				
1001300.0	3003.9000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0015	0.4300	0.6200				
108.0000	108.0000	0.00000001	0.0000	0.0025				
3847500.0	11542.5000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0015	0.4300	0.6200				
113.0000	113.0000	0.00000001	0.0000	0.0025				
2730800.0	8192.4000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0015	0.4300	0.6200				
113.0000	113.0000	0.00000001	0.0000	0.0025				
3059100.0	9177.3000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0015	0.4300	0.6200				
128.0000	128.0000	0.00000001	0.0000	0.0025				
2745800.0	8237.4000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0015	0.4300	0.6200				
115.0000	115.0000	0.00000001	0.0000	0.0025				
5713700.0	17141.1000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0015	0.4300	0.6200				
115.0000	115.0000	0.00000001	0.0000	0.0025				
15968800.0	47906.4000	0.0000	0.0000	0.0000	0.00000			
0.0000	0.0000	0.0015	0.4300	0.6200				
115.0000	115.0000	0.00000001	0.0000	0.0025				
124.8	3350.2							
1	1600010001000	1	0.005					

Input Files for DESRA-2C, Site Treasure Island (continued)

1200 5 0 1.0000

(5E15.7)

Loma Prieta 10/17/89, at Yerba Buena Island, N-S component [YBI000.A05]

.4751351E-03	.5475190E-03	.3324200E-03	-.2776069E-04	.2232370E-04
.2012119E-04	.1825253E-04	.1711811E-04	.1700532E-04	.1781620E-04
.1887424E-04	.1948468E-04	.1960467E-04	.1934467E-04	.1841196E-04
.1634096E-04	.1282801E-04	.7762477E-05	.1502838E-05	-.5460658E-05

(remainder of time history omitted)

Input Files for RASCAL, Site Treasure Island

A)

LOMA PRIETA AT TREASURE ISL, NS COMP, BASE-CASE PROFILE

NS

C:\freq\nrcL.frq

200,2.7,1.0,00.0,3.2,0.0

10000000.0,1.0,0.6,0.00,7.0,6.4,1.0,0.,0.,1.

5.0,0,0,0.000

2,0,0,0,-1,0,1,1,0

BASE3.PAR

..\..\MATERIAL\EPRITI4.MAT

..\..\DATA\YBI000.A05

7990 76 0.005 (5E15.7)

-1. -1. -1.

0 0

62.5 0 0.10 -0

..\..\WESSA\YBI000.FAS

4097 95(2E15.7)

B)

TRI FINAL BASE PROFILE, EPRI CURVES

1 44 44 10 0 0 3 1 44 0

.65 1.00 1.00 1.00 .05 0. 0. 0.0000

.00 .00 .00

1.25 175.25 1.920 1.000 2

1.25 175.25 1.920 1.000 2

1.375 133.50 1.920 1.000 2

1.375 133.50 1.920 1.000 2

1.375 133.50 1.920 1.000 2

1.375 133.50 1.920 1.000 2

1.64 177.85 1.920 1.000 2

1.64 177.85 1.920 1.000 2

1.402 177.85 1.920 1.000 2

1.402 177.85 1.770 0.900 3

1.402 177.85 1.770 0.900 3

1.402 177.85 1.770 0.900 3

1.402 177.85 1.770 0.900 3

1.52 207.25 1.770 0.900 3

1.52 207.25 1.770 0.900 3

1.494 164.58 1.770 0.900 3

Input Files for RASCAL, Site Treasure Island (continued)

B)

1.494	164.58	1.770	0.900	3
1.494	164.58	1.770	0.900	3
1.494	164.58	1.770	0.900	3
1.494	164.58	1.770	0.900	3
2.365	317.28	1.830	0.900	3
2.365	317.28	1.830	0.900	3
2.763	267.30	1.830	0.900	3
2.763	267.30	1.830	0.900	3
2.763	267.30	1.830	0.900	3
2.763	267.30	1.830	0.900	3
2.763	267.30	2.050	0.900	3
2.763	267.30	2.050	0.900	3
2.763	267.30	2.050	0.900	3
2.763	267.30	2.050	0.900	3
2.763	267.30	2.050	0.900	3
2.763	267.30	2.050	0.900	3
2.763	267.30	2.050	0.900	3
2.764	267.30	2.050	0.900	3
2.764	267.30	1.840	0.900	3
2.764	267.30	1.840	0.900	3
2.764	267.30	1.840	0.900	3
2.764	267.30	1.840	0.900	3
3.252	385.86	1.840	0.900	3
3.252	385.86	1.840	0.900	3
3.253	385.86	1.840	0.900	3
3.253	385.86	1.840	0.900	3
6.005	644.62	1.840	0.900	3
6.005	644.62	1.840	0.900	3
.000	1021.09	2.000	0.900	-1

C)

1. UNITY.MOD: CONSTANT MODULUS CURVE

1.0	1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.00	1.00						

1. UNITY DAMPING CURVE

1.0	1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.00	1.00						

2. EPRI SITING MOD RED FOR TREAS ISL, HYDROLIC FILL & SHOAL SANDS

Input Files for RASCAL, Site Treasure Island (continued)

C)

1.0	1.0	0.99	0.96	0.85	0.64	0.34	0.15
0.06	0.04						

2. EPRI SITING DAMPING FOR TREAS ISL, HYDROLIC FILL & SHOAL SANDS

1.0	1.0	1.2	1.7	3.1	6.0	12.4	19.2
22.8	25.0						

3. EPRI SITING MOD RED FOR TREAS ISL, ALL MUD & CLAY

1.0	1.0	1.00	0.99	0.95	0.85	0.63	0.38
0.18	0.10						

3. EPRI SITING DAMPING FOR TREAS ISL, ALL MUD & CLAY

0.9	0.9	1.0	1.1	1.6	2.7	5.4	9.7
13.5	16.0						

Input Files for SUMDES, Site Treasure Island

[Title of analysis]

SUMDES ver. 4.0: Treasure Island test case

[Global information: The basic units used in the file are m, kg, and sec]

101300.0 9.81 9810.0
2000.0 1021.09 2438.28
39 3 3

(Elastic base)

[Layer properties:]

0.5	2.65	18835.0	0.0E0	0.5	0.005	0.005	0.0E0	1
1.0	2.65	18835.0	0.0E0	0.5	0.005	0.005	0.0E0	1
1.0	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	1
1.0	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	2
1.0	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	2
1.0	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	2
1.0	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	2
1.5	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	2
1.0	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	3
1.0	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	3
1.0	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	3
1.0	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	3
1.1	2.65	18835.0	1.0E-5	0.5	0.005	0.005	2.2E9	3
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	4
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	4
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	4
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	4
1.2	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	4
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	5
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	5
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	5
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	6
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	6
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	6
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	6
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	6
1.0	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	6
1.5	2.7	17364.0	1.0E-8	0.75	0.005	0.005	2.2E9	6
4.7	2.7	17952.0	1.0E-8	0.75	0.005	0.005	2.2E9	7
6.1	2.7	17952.0	1.0E-8	0.75	0.005	0.005	2.2E9	8
6.1	2.7	17952.0	1.0E-8	0.75	0.005	0.005	2.2E9	8
8.5	2.7	20110.0	1.0E-8	0.75	0.005	0.005	2.2E9	9

Input Files for SUMDES, Site Treasure Island (continued)

8.5	2.7	20110.0	1.0E-8	0.75	0.005	0.005	2.2E9	9
6.1	2.7	18050.0	1.0E-8	0.75	0.005	0.005	2.2E9	10
6.1	2.7	18050.0	1.0E-8	0.75	0.005	0.005	2.2E9	10
6.5	2.7	18050.0	1.0E-8	0.75	0.005	0.005	2.2E9	11
6.5	2.7	18050.0	1.0E-8	0.75	0.005	0.005	2.2E9	11
6.0	2.7	18050.0	1.0E-8	0.75	0.005	0.005	2.2E9	12
6.0	2.7	18050.0	1.0E-8	0.75	0.005	0.005	2.2E9	12

(Two basic models are set for the sand and clay layers, respectively. The unit weight of the top layer is less because it is dry. The coefficient of permeability and the bulk modulus of pore fluid for the top layer are dummy values.)

[Model definition:]

```

1 4 40.0 459.0 2.3E-4 1.2E-3 1.399 1.0 0.75 0.2 2 35 /
2 4 40.0 130.0 2.4E-5 1.2E-4 0.203 1.0 0.75 0.2 2 35 /
3 4 40.0 165.0 3.8E-5 1.9E-4 0.264 1.0 0.75 0.2 2 35 /
4 4 30.0 323.0 1.4E-4 7.0E-4 2.166 100 0.75 100 2 35 /
5 4 30.0 414.0 1.5E-4 7.5E-4 2.220 100 0.75 100 2 35 /
6 4 30.0 210.0 1.8E-4 8.8E-4 0.882 100 0.75 100 2 35 /
7 4 30.0 511.0 1.0E-4 5.0E-4 4.713 100 0.75 100 2 35 /
8 4 30.0 294.0 1.2E-4 6.0E-4 2.374 100 0.75 100 2 35 /
9 4 30.0 121.0 1.0E-4 5.1E-4 1.978 100 0.75 100 2 35 /
10 4 30.0 184.0 1.4E-4 7.1E-4 1.642 100 0.75 100 2 35 /
11 4 30.0 353.0 1.4E-4 7.0E-4 3.868 100 0.75 100 2 35 /
12 4 30.0 929.0 8.8E-5 4.4E-4 13.07 100 0.75 100 2 35 /

```

(Model ID No. 1 - 12: Use the reduced-order bounding surface model Model 4)

[Input acceleration:]

```

0.0 0.005 7000
1.0 1.0 1.0
...\DATA\YBI000.A05
76
(5E15.7)
DUMMY
0
Dummy format
DUMMY
0
Dummy format

```

(Only horizontal direction 1 is active)

Input Files for SUMDES, Site Treasure Island (continued)

[Numerical control parameters]

0.0 0.25 0.5
0.001 5 16 0.1 5.0

[Output files]

ti-4NS.OUT
4 0.175
ti-4NS.sum

Input Files for TESS, Site Treasure Island

TREASURE ISLAND, LOMA PRIETA, NORTH-SOUTH
BASE CASE PROFILE

C NLAYERS NUMMAT KDIS NRUN
18 8 1 1

C PATM DEPTH SLOPE
2.116 4. 0.

C NLAYOUT DTIME
1 2.

C MATERIAL PROPERTIES

C MTYPE KGAT KDEG KCON VG VT
1 -2 0 3 0.02 0.02

C MTYPE KGAT KDEG KCON VG VT
2 2 2 3 0.02 0.02

C KG NG NOCR KTMAX
1265 0.5 1 1

C C PHI DPHI
.1 56 0

C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
3 2 2 3 0.02 0.02

C KG NG NOCR KTMAX
604 0.5 1 1

C C PHI DPHI
.1 28 0

C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
4 2 2 3 0.02 0.02

C KG NG NOCR KTMAX
513 0.5 1 1

C C PHI DPHI
.1 21 0

C E F G Code
2. .1 2. 0

Input Files for TESS, Site Treasure Island (continued)

```
C MTYPE  KGAT  KDEG  KCON  VG  VT
  5  2  2  3  0.02 0.02
C KG NG NOCR KTMAX
  451 0.5 1 1
C C PHI DPHI
  .1 16 0
C E F G Code
  2. .1 2. 0
```

```
C MTYPE  KGAT  KDEG  KCON  VG  VT
  6  2  2  3  0.02 0.02
C KG NG NOCR KTMAX
  722 0.5 1 1
C C PHI DPHI
  .1 25 0
C E F G Code
  2. .1 2. 0
```

```
C MTYPE  KGAT  KDEG  KCON  VG  VT
  7  2  2  3  0.02 0.02
C KG NG NOCR KTMAX
  644 0.5 1 1
C C PHI DPHI
  .1 20 0
C E F G Code
  2. .1 2. 0
```

```
C MTYPE  KGAT  KDEG  KCON  VG  VT
  8  -2  1  0 0.02 0.02
C ss rs e g sg rg st rt
  0.12 0.65 1.5 .5 .12 .65 .12 .65
```

C LAYER DATA

```
C No. Matype Thick. Unit Wt.
  1 1 4. .1198
C Vs GAMr
  575. 0.08
C MV K
  0.001 0.0005
```

Input Files for TESS, Site Treasure Island (continued)

C No. Matype Thick. Unit Wt.

2 2 4.2 .1198

C Ko OCR

0.5 1.0

C CSR NI

.10 5

C MV K

0.001 0.0005

C No. Matype Thick. Unit Wt.

3 3 6. .1198

C Ko OCR

0.5 1.0

C CSR NI

.10 5

C MV K

0.001 0.0005

C No. Matype Thick. Unit Wt.

4 4 6. .1198

C Ko OCR

0.5 1.0

C CSR NI

.10 5

C MV K

0.001 0.0005

C No. Matype Thick. Unit Wt.

5 5 6. .1198

C Ko OCR

0.5 1.0

C CSR NI

.10 5

C MV K

0.001 0.0005

C No. Matype Thick. Unit Wt.

6 6 8. .1198

C Ko OCR

0.5 1.0

C CSR NI

.10 5

C MV K

0.001 0.0005

Input Files for TESS, Site Treasure Island (continued)

C No. Matype Thick. Unit Wt.
7 7 8.8 .1198
C Ko OCR
0.5 1.0
C CSR NI
.10 5
C MV K
0.001 0.0005

C No. Matype Thick. Unit Wt.
8 8 8. .1104
C Vs GAMr
584. 0.3

C No. Matype Thick. Unit Wt.
9 8 9. .1104
C Vs GAMr
584. 0.3

C No. Matype Thick. Unit Wt.
10 8 10. .1104
C Vs GAMr
680. 0.3

C No. Matype Thick. Unit Wt.
11 8 12. .1104
C Vs GAMr
540. 0.3

C No. Matype Thick. Unit Wt.
12 8 12.5 .1104
C Vs GAMr
540. 0.3

C No. Matype Thick. Unit Wt.
13 8 15.5 .1142
C Vs GAMr
1041. 0.3

C No. Matype Thick. Unit Wt.
14 8 40. .1142
C Vs GAMr
877. 0.3

Input Files for TESS, Site Treasure Island (continued)

C No. Matype Thick. Unit Wt.

15 8 56. .1279

C Vs GAMr

877. 0.3

C No. Matype Thick. Unit Wt.

16 8 40. .1148

C Vs GAMr

877. 0.3

C No. Matype Thick. Unit Wt.

17 8 42.7 .1148

C Vs GAMr

1266. 0.3

C No. Matype Thick. Unit Wt.

18 8 39.4 .1148

C Vs GAMr

2115. 0.3

C COMPLIANT BASE DATA

C Vs Unit Wt. KBASE

3350 .1248 1

C OUTPUT OPTIONS

C Layer No. No. of histories

1 3

C Codes for type of histories

1 4 5

C INPUT MOTION

LOMA PRIETA AT YERBA BUENA ISLAND, NORTH-SOUTH

C Code Code NPTS NTOT DT Amax KREAD

1 0 7990 7990 .005 0.02885 5

.4751351E-03	.5475190E-03	.3324200E-03	-.2776069E-04	.2232370E-04
.2012119E-04	.1825253E-04	.1711811E-04	.1700532E-04	.1781620E-04
.1887424E-04	.1948468E-04	.1960467E-04	.1934467E-04	.1841196E-04
.1634096E-04	.1282801E-04	.7762477E-05	.1502838E-05	-.5460658E-05

(remainder of time history omitted)

Input Files for DESRA-2C, RASCAL, SUMDES, and TESS for the Lotung Analyses

DESRA-2: LOTUNG LSST Event 07, input motion at 47 m

0.3520 1.1350 0.0033 0.4300 0.6200

Input Files for DESRA-2C, Site LSST, Event 07 (continued)

112.0000	112.0000	0.0001	0.0000	0.0041	
1285000.0	2180.0000	0.0000	0.0000	0.0000	0.00000
0.4890	0.8180	0.0035	0.4300	0.6200	.
112.0000	112.0000	0.0001	0.0000	0.0041	
1340000.0	2284.0000	0.0000	0.0000	0.0000	0.00000
0.4890	0.8180	0.0035	0.4300	0.6200	
112.0000	112.0000	0.0001	0.0000	0.0041	
2070000.0	3524.0000	0.0000	0.0000	0.0000	0.00000
0.4890	0.8180	0.0035	0.4300	0.6200	
112.0000	112.0000	0.0001	0.0000	0.0041	
2230000.0	3787.0000	0.0000	0.0000	0.0000	0.00000
0.4890	0.8180	0.0035	0.4300	0.6200	
112.0000	112.0000	0.0001	0.0000	0.0041	
1840000.0	3125.0000	0.0000	0.0000	0.0000	0.00000
0.4890	0.8180	0.0035	0.4300	0.6200	
112.0000	112.0000	0.0001	0.0000	0.0041	
1940000.0	3304.0000	0.0000	0.0000	0.0000	0.00000
0.4890	0.8180	0.0035	0.4300	0.6200	
112.0000	112.0000	0.0001	0.0000	0.0041	
3820000.0	6487.0000	0.0000	0.0000	0.0000	0.00000
0.0000	0.0000	0.0035	0.4300	0.6200	
112.0000	112.0000	0.0001	0.0000	0.0041	
2300000.0	3910.0000	0.0000	0.0000	0.0000	0.00000
0.0000	0.0000	0.0035	0.4300	0.6200	
112.0000	112.0000	0.0001	0.0000	0.0041	
2520000.0	4277.0000	0.0000	0.0000	0.0000	0.00000
0.0000	0.0000	0.0035	0.4300	0.6200	
112.0000	112.0000	0.0001	0.0000	0.0041	
2360000.0	4010.0000	0.0000	0.0000	0.0000	0.00000
0.0000	0.0000	0.0035	0.4300	0.6200	
112.0000	112.0000	0.0001	0.0000	0.0041	
2460000.0	4174.0000	0.0000	0.0000	0.0000	0.00000
0.0000	0.0000	0.0035	0.4300	0.6200	
112.0000	112.0000	0.0001	0.0000	0.0041	
1	1700010001000	1	0.005		

1417 5 0 0.00102

(5E12.5)

LOTUNG LSST Event 07, at DHB47, N-S component [db4707ns.acc]

.000000E+00	.000000E+00	.000000E+00	.000000E+00	.000000E+00
.000000E+00	.000000E+00	.000000E+00	.000000E+00	.000000E+00
.000000E+00	.000000E+00	.000000E+00	.000000E+00	.000000E+00

(remainder of time history omitted)

Input Files for DESRA-2C, Site LSST, Event 16

DESRA-2: LOTUNG LSST Event 16, input motion at 17 m

3	13	7	0	0	0	2	0	1	1	1	0
1	2		3.90			1.0	0				
1	2		3.90			1.0	0				
2	2		5.80			1.0	0				
3	2		4.20			1.0	0				
3	2		4.20			1.0	0				
4	2		3.75			1.0	0				
4	2		3.75			1.0	0				
5	2		3.70			1.0	0				
5	2		3.70			1.0	0				
6	2		4.95			1.0	0				
6	2		4.95			1.0	0				
7	2		5.00			1.0	0				
7	2		3.97			1.0	0				
484000.0		823.0000		0.0000		0.0000		0.0000		0.00000	
0.2720		1.4700		0.0022		0.4300		0.6200			
112.0000		112.0000		0.0001		0.0000		0.0041			
546000.0		929.0000		0.0000		0.0000		0.0000		0.00000	
0.2720		1.4700		0.0022		0.4300		0.6200			
112.0000		112.0000		0.0001		0.0000		0.0041			
836000.0		142.0000		0.0000		0.0000		0.0000		0.00000	
0.2720		1.4700		0.0022		0.4300		0.6200			
112.0000		112.0000		0.0001		0.0000		0.0041			
1020000.0		1727.0000		0.0000		0.0000		0.0000		0.00000	
0.3520		1.1350		0.0033		0.4300		0.6200			
112.0000		112.0000		0.0001		0.0000		0.0041			
1210000.0		2063.0000		0.0000		0.0000		0.0000		0.00000	
0.3520		1.1350		0.0033		0.4300		0.6200			
112.0000		112.0000		0.0001		0.0000		0.0041			
1285000.0		2180.0000		0.0000		0.0000		0.0000		0.00000	
0.4890		0.8180		0.0035		0.4300		0.6200			
112.0000		112.0000		0.0001		0.0000		0.0041			
1340000.0		2284.0000		0.0000		0.0000		0.0000		0.00000	
0.4890		0.8180		0.0035		0.4300		0.6200			
112.0000		112.0000		0.0001		0.0000		0.0041			
1	1999910001000	1		0.005							

2237 5 0 0.00102
(5E12.5)

LOTUNG LSST Event 16, at DHB17, N-S component [db1716NS.acc]

.00000E+00 .00000E+00 .00000E+00 .00000E+00 .00000E+00
 .00000E+00 .00000E+00 .00000E+00 .00000E+00 .00000E+00
 .00000E+00 .00000E+00 .00000E+00 .00000E+00 .00000E+00
 .00000E+00 .00000E+00 .00000E+00 .00000E+00 .00000E+00
 (remainder of time history omitted)

Input Files for RASCAL, Site LSST, Event 07

A)

EVENT 07 AT LSST 00M, NS COMP, INPUT 47M, BASE-CASE PROFILE
NS00S

C:\freq\nrcL.frq

300,2.7,1.0,00.0,3.2,0.0

10000000.0,1.0,0.6,0.00,6.5,0.0,0.00102,0.,0.,1.

5.0,0,0,0.000

2,0,0,0,-1,0,1,1,0

BASE0047.PAR

..\..\MATERIAL\EPRILT2.MAT

..\..\DATA\DB4707NS.ACC

7080 8 0.005 (5E12.5)

-1. -1. -1.

0 0

62.5 0 0.10 -0

..\..\WESSA\EV07\DB4707NS.FAS

4097 27(2E15.7)

B)

LOTUNG S-VEL BASE CASE, INPUT 47M OUTPUT 00M, EPRI CURVES

1 33 33 7 -1 0 2 1 33 0

.65 1.00 1.00 1.00 .05 0.0 0. 0.

.00 .00 .00

1.180 113.730 1.80 1.000 2

1.180 113.730 1.80 1.000 2

1.500 120.780 1.80 1.000 2

0.270 120.780 1.80 1.000 2

1.370 149.410 1.80 1.000 2

0.500 149.410 1.80 1.000 2

0.680 149.410 1.80 1.000 2

1.260 164.710 1.85 1.000 2

1.020 164.710 1.85 1.000 2

2.040 180.000 1.85 1.000 2

0.200 180.000 1.80 1.000 2

1.510 185.100 1.80 1.000 2

1.510 185.100 1.80 1.000 2

1.390 189.410 1.80 1.000 2

1.390 189.410 1.80 1.000 2

Input Files for RASCAL, Site LSST, Event 07

B)

0.290	189.410	2.00	1.000	2
2.550	235.290	2.00	1.000	2
2.260	243.920	2.00	1.000	2
1.590	243.920	1.90	1.000	2
1.810	221.570	1.90	1.000	2
1.945	227.840	1.90	1.000	2
1.945	227.840	1.90	1.000	2
0.410	319.220	1.90	1.000	2
2.035	319.220	1.90	1.000	2
2.035	319.220	1.90	1.000	2
1.930	247.840	1.90	1.000	2
1.140	247.840	1.90	1.000	2
2.080	259.220	1.80	1.000	2
1.905	250.980	1.80	1.000	2
1.905	250.980	1.80	1.000	2
2.085	256.080	2.00	1.000	2
2.085	256.080	2.00	1.000	2
2.510	256.080	2.00	1.000	2

C)

1. UNITY.MOD: CONSTANT MODULUS CURVE

1.0	1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.00	1.00						

1. UNITY DAMPING CURVE

1.0	1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.00	1.00						

2. EPRI SITING MODULUS REDUCTION FOR LSST, FINAL PYKE CURVE

1.0	1.000	1.00	0.97	0.87	0.69	0.42	0.210
0.09	0.05						

2. EPRI SITING DAMPING FOR LSST, FINAL PYKE CURVE

1.0	1.0	1.1	1.5	2.80	5.3	10.5	16.8
22.1	26.0						

Input Files for RASCAL, Site LSST, Event 16

A)

EVENT 16 AT LSST 00M, NS COMP, INPUT 17M, BASE-CASE PROFILE
NS00

C:\freq\nrcL.frq
100,2.7,1.0,00.0,3.2,0.0
10000000.0,1.0,0.6,0.00,7.8,0.0,0.00102,0.,0.,1.
5.0,0,0,0.000
2,0,0,0,-1,0,1,1,0
BASE0017.PAR
..\..\MATERIAL\EPRILT2.MAT
..\..\DATA\DB1716NS.ACC
10000 8 0.005 (5E12.5)
-1. -1. -1.
0 0
62.5 0 0.10 -0
..\..\WESSA\EV16\DB1716NS.FAS
4097 27(2E15.7)

B)

LOTUNG S-VEL BASE CASE, INPUT 17M OUTPUT 00M, EPRI CURVES

1 16 16 7 -1 0 2 1 17 0
.65 1.00 1.00 1.00 .05 0.0 0. 0.
.00 .00 .00
1.180 113.730 1.80 1.000 2
1.180 113.730 1.80 1.000 2
1.500 120.780 1.80 1.000 2
0.270 120.780 1.80 1.000 2
1.370 149.410 1.80 1.000 2
0.500 149.410 1.80 1.000 2
0.680 149.410 1.80 1.000 2
1.260 164.710 1.85 1.000 2
1.020 164.710 1.85 1.000 2
2.040 180.000 1.85 1.000 2
0.200 180.000 1.80 1.000 2
1.510 185.100 1.80 1.000 2
1.510 185.100 1.80 1.000 2
1.390 189.410 1.80 1.000 2
1.390 189.410 1.80 1.000 2

Input Files for RASCAL, Site LSST, Event 16 (continued)

B)

0.290	189.410	2.00	1.000	2
2.550	235.290	2.00	1.000	2
2.260	243.920	2.00	1.000	2
1.590	243.920	1.90	1.000	2
1.810	221.570	1.90	1.000	2
1.945	227.840	1.90	1.000	2
1.945	227.840	1.90	1.000	2
0.410	319.220	1.90	1.000	2
2.035	319.220	1.90	1.000	2
2.035	319.220	1.90	1.000	2
1.930	247.840	1.90	1.000	2
1.140	247.840	1.90	1.000	2
2.080	259.220	1.80	1.000	2
1.905	250.980	1.80	1.000	2
1.905	250.980	1.80	1.000	2
2.085	256.080	2.00	1.000	2
2.085	256.080	2.00	1.000	2
2.510	256.080	2.00	1.000	2

C)

1. UNITY.MOD: CONSTANT MODULUS CURVE

1.0	1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.00	1.00						

1. UNITY DAMPING CURVE

1.0	1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.00	1.00						

2. EPRI SITING MODULUS REDUCTION FOR LSST, FINAL PYKE CURVE

1.0	1.000	1.00	0.97	0.87	0.69	0.42	0.210
0.09	0.05						

2. EPRI SITING DAMPING FOR LSST, FINAL PYKE CURVE

1.0	1.0	1.1	1.5	2.80	5.3	10.5	16.8
22.1	26.0						

Input Files for SUMDES, Site LSST, Event 07

[Title of analysis]

SUMDES ver. 4.0: Lotung test case

[Global information: The basic units used in the file are m, kg, and sec]

101300.0 9.81 9810.0

2500.0 1.0E10 1.0E10

24 1 1

(Rigid base, input motion from Event 07 (5/20/86) recorded at 47m)

[Layer properties:]

1.0	2.65	17658.0	1.0E-5	0.80	0.01	0.005	3.0E8	1
1.36	2.65	17658.0	2.0E-6	0.80	0.01	0.005	3.0E8	1
1.77	2.65	17658.0	2.0E-6	1.0	0.01	0.005	6.0E8	2
1.37	2.65	17658.0	2.0E-6	1.0	0.01	0.005	9.0E8	3
1.18	2.65	17658.0	2.0E-6	1.0	0.01	0.005	3.0E9	4
1.14	2.65	18148.0	2.0E-6	1.0	0.01	0.005	3.0E9	5
1.14	2.65	18148.0	1.0E-7	1.0	0.01	0.005	3.0E9	5
1.12	2.65	18148.0	1.0E-7	1.0	0.01	0.005	3.0E9	6
1.12	2.65	18148.0	1.0E-5	1.0	0.01	0.005	3.0E9	6
1.51	2.65	17658.0	1.0E-5	1.0	0.01	0.005	3.0E9	7
1.51	2.65	17658.0	1.0E-5	1.0	0.01	0.005	3.0E9	7
1.53	2.7	17658.0	1.0E-7	1.0	0.01	0.005	3.0E9	8
1.53	2.7	17658.0	1.0E-7	1.0	0.01	0.005	3.0E9	8
2.55	2.65	19620.0	1.0E-5	1.0	0.01	0.005	3.0E9	9
2.26	2.65	19620.0	1.0E-5	1.0	0.01	0.005	3.0E9	10
1.59	2.7	18639.0	1.0E-7	1.0	0.01	0.005	3.0E9	11
1.81	2.65	18639.0	1.0E-5	1.0	0.01	0.005	3.0E9	12
3.89	2.65	18639.0	1.0E-5	1.0	0.01	0.005	3.0E9	13
4.48	2.65	18639.0	1.0E-5	1.0	0.01	0.005	3.0E9	14
3.07	2.7	18639.0	1.0E-7	1.0	0.01	0.005	3.0E9	15
2.08	2.7	17658.0	1.0E-7	1.0	0.01	0.005	3.0E9	16
3.81	2.7	17658.0	1.0E-7	1.0	0.01	0.005	3.0E9	17
2.09	2.7	19620.0	1.0E-7	1.0	0.01	0.005	3.0E9	18
2.09	2.7	19620.0	1.0E-7	1.0	0.01	0.005	3.0E9	18

(The water table is located at the surface)

[Model definition:]

1	4	64.0	306.0	0.0020	0.0004	0.247	4.0	0.75	0.2	2.0	35.0	/
2	4	40.0	194.0	0.0137	0.0027	0.245	3.5	0.75	0.2	2.0	35.0	/
3	4	42.0	244.0	0.0109	0.0022	0.305	3.5	0.75	0.2	2.0	35.0	/
4	4	35.0	217.0	0.0123	0.0025	0.336	3.5	0.75	0.2	2.0	35.0	/

Input Files for SUMDES, Site LSST, Event 07 (continued)

5	4	36.0	196.0	0.0107	0.0021	0.360	3.5	0.75	0.2	2.0	35.0	/
6	4	34.0	208.0	0.0101	0.0020	0.424	3.5	0.75	0.2	2.0	35.0	/
7	4	29.0	230.0	0.0116	0.0023	0.485	3.5	0.75	0.2	2.0	35.0	/
8	4	25.0	237.0	0.0127	0.0025	0.572	3.5	0.75	0.2	2.0	35.0	/
9	4	35.0	190.0	0.0068	0.0013	0.626	3.5	0.75	0.2	2.0	35.0	/
10	4	33.0	179.0	0.0065	0.0013	0.694	3.5	0.75	0.2	2.0	35.0	/
11	4	29.0	242.0	0.0078	0.0016	0.800	5.0	0.75	0.2	2.0	35.0	/
12	4	23.0	176.0	0.0095	0.0019	0.833	3.5	0.75	0.2	2.0	35.0	/
13	4	22.0	176.0	0.0095	0.0019	0.912	3.5	0.75	0.2	2.0	35.0	/
14	4	35.0	321.0	0.0052	0.0010	1.025	3.5	0.75	0.2	2.0	35.0	/
15	4	20.0	199.0	0.0095	0.0019	1.095	5.0	0.75	0.2	2.0	35.0	/
16	4	20.0	290.0	0.0104	0.0021	1.142	5.0	0.75	0.2	2.0	35.0	/
17	4	17.0	138.0	0.0218	0.0044	0.468	5.0	0.75	0.2	2.0	35.0	/
18	4	18.0	144.0	0.0089	0.0018	1.221	5.0	0.75	0.2	2.0	35.0	/

(Model ID No. 1 - 18: Use the reduced-order bounding surface model
Model 4)

[Input acceleration:]

```
0.0 0.005 6000
0.001019 1.0 1.0
..\..\data\db4707ns.ACC
8
(5E12.5)
DUMMY
0
Dummy format
DUMMY
0
Dummy format
```

(Only horizontal direction 1 is active)

[Numerical control parameters]

```
0.0 0.25 0.5
0.001 5 16 0.1 5.0
```

[Output files]

```
lt07-1ns.out
4 0.20
lt07-1ns.sum
```


Input Files for SUMDES, Site LSST, Event 16

[Title of analysis]

SUMDES ver. 4.0: Lotung test case

[Global information: The basic units used in the file are m, kg, and sec]

101300.0 9.81 9810.0

2500.0 1.0E10 1.0E10

13 1 1

(Rigid base, input motion from Event 16 (11/14/86) recorded at 17m)

[Layer properties:]

1.0	2.65	17658.0	1.0E-5	0.80	0.01	0.005	3.0E8	1
1.36	2.65	17658.0	2.0E-6	0.80	0.01	0.005	3.0E8	1
1.77	2.65	17658.0	2.0E-6	1.0	0.01	0.005	6.0E8	2
1.37	2.65	17658.0	2.0E-6	1.0	0.01	0.005	9.0E8	3
1.18	2.65	17658.0	2.0E-6	1.0	0.01	0.005	3.0E9	4
1.14	2.65	18148.0	2.0E-6	1.0	0.01	0.005	3.0E9	5
1.14	2.65	18148.0	1.0E-7	1.0	0.01	0.005	3.0E9	5
1.12	2.65	18148.0	1.0E-7	1.0	0.01	0.005	3.0E9	6
1.12	2.65	18148.0	1.0E-5	1.0	0.01	0.005	3.0E9	6
1.51	2.65	17658.0	1.0E-5	1.0	0.01	0.005	3.0E9	7
1.51	2.65	17658.0	1.0E-5	1.0	0.01	0.005	3.0E9	7
1.53	2.7	17658.0	1.0E-7	1.0	0.01	0.005	3.0E9	8
1.53	2.7	17658.0	1.0E-7	1.0	0.01	0.005	3.0E9	8

(The water table is located at the surface)

[Model definition:]

1	4	64.0	306.0	0.0020	0.0004	0.247	4.0	0.75	0.2	2.0	35.0	/
2	4	40.0	194.0	0.0137	0.0027	0.245	3.5	0.75	0.2	2.0	35.0	/
3	4	42.0	244.0	0.0109	0.0022	0.305	3.5	0.75	0.2	2.0	35.0	/
4	4	35.0	217.0	0.0123	0.0025	0.336	3.5	0.75	0.2	2.0	35.0	/
5	4	36.0	196.0	0.0107	0.0021	0.360	3.5	0.75	0.2	2.0	35.0	/
6	4	34.0	208.0	0.0101	0.0020	0.424	3.5	0.75	0.2	2.0	35.0	/
7	4	29.0	230.0	0.0116	0.0023	0.485	3.5	0.75	0.2	2.0	35.0	/
8	4	25.0	237.0	0.0127	0.0025	0.572	3.5	0.75	0.2	2.0	35.0	/

(Model ID No. 1 - 8: Use the reduced-order bounding surface model
Model 4)

Input Files for SUMDES, Site LSST, Event 16 (continued)

[Input acceleration:]

0.0 0.005 10000
0.001019 1.0 1.0
...\data\db1716ns.ACC
8
(5E12.5)
DUMMY
0
Dummy format
DUMMY
0
Dummy format

(Only horizontal direction 1 is active)

[Numerical control parameters]

0.0 0.25 0.5
0.001 5 16 0.1 5.0

[Output files]

lt16-1ns.out
4 0.20
lt16-1ns.sum

Input Files for TESS, Site LSST, Event 07

LOTUNG LSST EVENT 07, 47M INPUT, NORTH-SOUTH
BASE CASE PROFILE

C NLAYERS NUMMAT KDIS NRUN
18 18 1 1

C PATM DEPTH SLOPE
2.116 0. 0.

C NLAYOUT DTIME
4 2.

C MATERIAL PROPERTIES

C MTYPE KGAT KDEG KCON VG VT
1 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
893 0.5 1 1
C C PHI DPHI
.1 64 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
2 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
596 0.5 1 1
C C PHI DPHI
.1 40 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
3 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
733 0.5 1 1
C C PHI DPHI
.1 42 0
C E F G Code
2. .1 2. 0

Input Files for TESS, Site LSST, Event 07 (continued)

```
C MTYPE  KGAT  KDEG  KCON  VG  VT
  4  2  2  3  0.02 0.02
C KG  NG  NOCR  KTMAX
  653 0.5  1  1
C C  PHI  DPHI
  .1 35  0
C  E F  G Code
  2. .1  2. 0
```

```
C MTYPE  KGAT  KDEG  KCON  VG  VT
  5  2  2  3  0.02 0.02
C KG  NG  NOCR  KTMAX
  732 0.5  1  1
C C  PHI  DPHI
  .1 36  0
C  E F  G Code
  2. .1  2. 0
```

```
C MTYPE  KGAT  KDEG  KCON  VG  VT
  6  2  2  3  0.02 0.02
C KG  NG  NOCR  KTMAX
  767 0.5  1  1
C C  PHI  DPHI
  .1 34  0
C  E F  G Code
  2. .1  2. 0
```

```
C MTYPE  KGAT  KDEG  KCON  VG  VT
  7  2  2  3  0.02 0.02
C KG  NG  NOCR  KTMAX
  700 0.5  1  1
C C  PHI  DPHI
  .1 29  0
C  E F  G Code
  2. .1  2. 0
```

Input Files for TESS, Site LSST, Event 07 (continued)

C MTYPE KGAT KDEG KCON VG VT
8 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
661 0.5 1 1
C C PHI DPHI
.1 25 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
9 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
1035 0.5 1 1
C C PHI DPHI
.1 35 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
10 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
1036 0.5 1 1
C C PHI DPHI
.1 33 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
11 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
935 0.5 1 1
C C PHI DPHI
.1 29 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
12 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
744 0.5 1 1
C C PHI DPHI
.1 23 0
C E F G Code
2. .1 2. 0

Input Files for TESS, Site LSST, Event 07 (continued)

C MTYPE KGAT KDEG KCON VG VT
13 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
742 0.5 1 1
C C PHI DPHI
.1 22 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
14 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
1353 0.5 1 1
C C PHI DPHI
.1 35 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
15 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
770 0.5 1 1
C C PHI DPHI
.1 20 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
16 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
770 0.5 1 1
C C PHI DPHI
.1 20 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
17 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
698 0.5 1 1
C C PHI DPHI
.1 17 0
C E F G Code
2. .1 2. 0

Input Files for TESS, Site LSST, Event 07 (continued)

C MTYPE KGAT KDEG KCON VG VT
18 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
769 0.5 1 1
C C PHI DPHI
.1 18 0
C E F G Code
2. .1 2. 0

C LAYER DATA

C No. Matype Thick. Unit Wt.
1 1 7.74 .1123
C Ko OCR
0.5 1.0
C CSR NI
.30 5
C MV K
0.001 0.0001

C No. Matype Thick. Unit Wt.
2 2 5.81 .1123
C Ko OCR
0.5 1.0
C CSR NI
.30 5
C MV K
0.001 0.0001

C No. Matype Thick. Unit Wt.
3 3 6.13 .1123
C Ko OCR
0.5 1.0
C CSR NI
.30 5
C MV K
0.001 0.0001

Input Files for TESS, Site LSST, Event 07 (continued)

C No. Matype Thick. Unit Wt.

4 4 2.23 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

5 5 7.48 .1154

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

6 6 7.35 .1154

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

7 7 9.91 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

8 8 10.07 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

Input Files for TESS, Site LSST, Event 07 (continued)

C No. Matype Thick. Unit Wt.

9 9 8.37 .1248

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

10 10 7.42 .1248

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

11 11 5.22 .1186

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

12 12 5.94 .1186

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

13 13 12.76 .1186

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

Input Files for TESS, Site LSST, Event 07 (continued)

C No. Matype Thick. Unit Wt.

14 14 14.70 .1186

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

15 15 10.07 .1186

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

16 16 6.82 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

17 17 12.50 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

18 18 13.68 .1248

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

Input Files for TESS, Site LSST, Event 07 (continued)

C COMPLIANT BASE DATA

C Vs Unit Wt. KBASE

84000 .1248 1

C OUTPUT OPTIONS

C Layer No. No. of histories

1 3

C Codes for type of histories

1 4 5

C Layer No. No. of histories

4 3

C Codes for type of histories

1 4 5

C Layer No. No. of histories

7 3

C Codes for type of histories

1 4 5

C Layer No. No. of histories

9 3

C Codes for type of histories

1 4 5

C INPUT MOTION

EVENT 07 47 M DEPTH, NORTH-SOUTH

C Code Code NPTS NTOT DT Amax KREAD

1 0 10000 10000 .0025 0.0989 5

.0000000E+00 .0000000E+00 .0000000E+00 .0000000E+00 .0000000E+00

.0000000E+00 .0000000E+00 .0000000E+00 .0000000E+00 .0000000E+00

.0000000E+00 .0000000E+00 .0000000E+00 .0000000E+00 .0000000E+00

.0000000E+00 .0000000E+00 .0000000E+00 .0000000E+00 .0000000E+00

(remainder of time history omitted)

Input Files for TESS, Site LSST, Event 16

LOTUNG LSST EVENT 16, 17M INPUT, NORTH-SOUTH
BASE CASE PROFILE

C NLAYERS NUMMAT KDIS NRUN
8 8 1 1

C PATM DEPTH SLOPE
2.116 0. 0.

C NLAYOUT DTIME
3 2.

C MATERIAL PROPERTIES

C MTYPE KGAT KDEG KCON VG VT
1 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
893 0.5 1 1
C C PHI DPHI
.1 64 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
2 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
596 0.5 1 1
C C PHI DPHI
.1 40 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
3 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
733 0.5 1 1
C C PHI DPHI
.1 42 0
C E F G Code
2. .1 2. 0

Input Files for TESS, Site LSST, Event 16 (continued)

C MTYPE KGAT KDEG KCON VG VT
4 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
653 0.5 1 1
C C PHI DPHI
.1 35 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
5 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
732 0.5 1 1
C C PHI DPHI
.1 36 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
6 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
767 0.5 1 1
C C PHI DPHI
.1 34 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
7 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
700 0.5 1 1
C C PHI DPHI
.1 29 0
C E F G Code
2. .1 2. 0

C MTYPE KGAT KDEG KCON VG VT
8 2 2 3 0.02 0.02
C KG NG NOCR KTMAX
661 0.5 1 1
C C PHI DPHI
.1 25 0
C E F G Code
2. .1 2. 0

Input Files for TESS, Site LSST, Event 16 (continued)

C LAYER DATA

C No. Matype Thick. Unit Wt.

1 1 7.74 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

2 2 5.81 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

3 3 6.13 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

4 4 2.23 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

Input Files for TESS, Site LSST, Event 16 (continued)

C No. Matype Thick. Unit Wt.

5 5 7.48 .1154

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

6 6 7.35 .1154

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

7 7 9.91 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C No. Matype Thick. Unit Wt.

8 8 10.07 .1123

C Ko OCR

0.5 1.0

C CSR NI

.30 5

C MV K

0.001 0.0001

C COMPLIANT BASE DATA

C Vs Unit Wt. KBASE

84000 .1248 1

Input Files for TESS, Site LSST, Event 16 (continued)

C OUTPUT OPTIONS

C Layer No. No. of histories

1 3

C Codes for type of histories

1 4 5

C Layer No. No. of histories

4 3

C Codes for type of histories

1 4 5

C Layer No. No. of histories

7 3

C Codes for type of histories

1 4 5

C INPUT MOTION

EVENT 16 17 M DEPTH, NORTH-SOUTH

C	Code	Code	NPTS	NTOT	DT	Amax	KREAD

1	0	10000	10000	.0025	0.086	5	

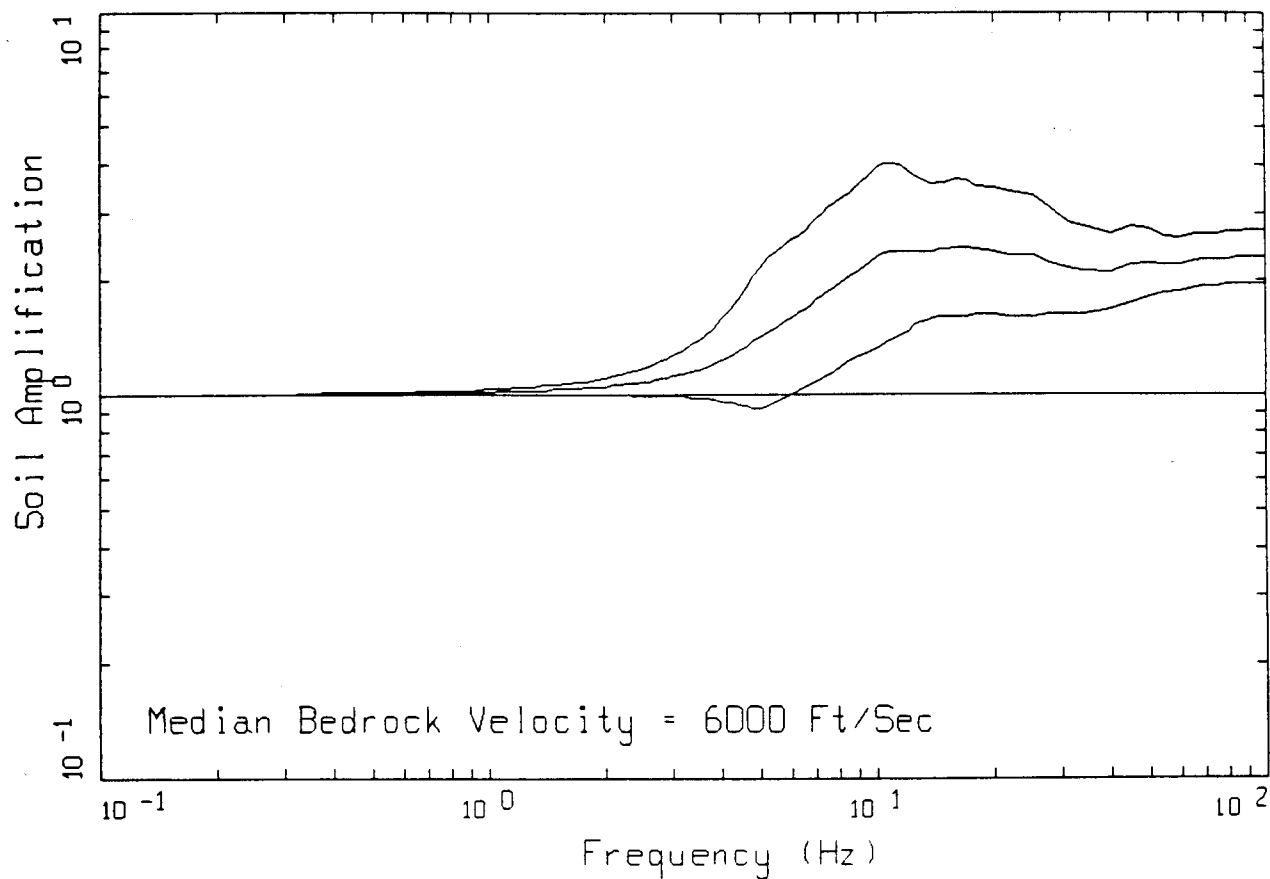
.6320000E+00	.7399794E+00	.8480000E+00	.9154871E+00	.9830000E+00
.1100478E+01	.1218000E+01	.1510444E+01	.1803000E+01	.2279500E+01
.2756000E+01	.3305605E+01	.3855000E+01	.4361096E+01	.4867000E+01
.5305084E+01	.5743000E+01	.6176917E+01	.6611000E+01	.7121902E+01

(remainder of time history omitted)

APPENDIX 6.C

PLOTS OF AMPLIFICATION FACTORS

Plots of 5% Damped Response Spectral Amplification Factors (SA (Soil)/SA (Rock)) for the Five Site Categories (Table 6-2) and 8 Levels of Input Motions (Table 6-3). The Figure Legends Indicate the Control Motion Outcrop Peak Acceleration Levels.

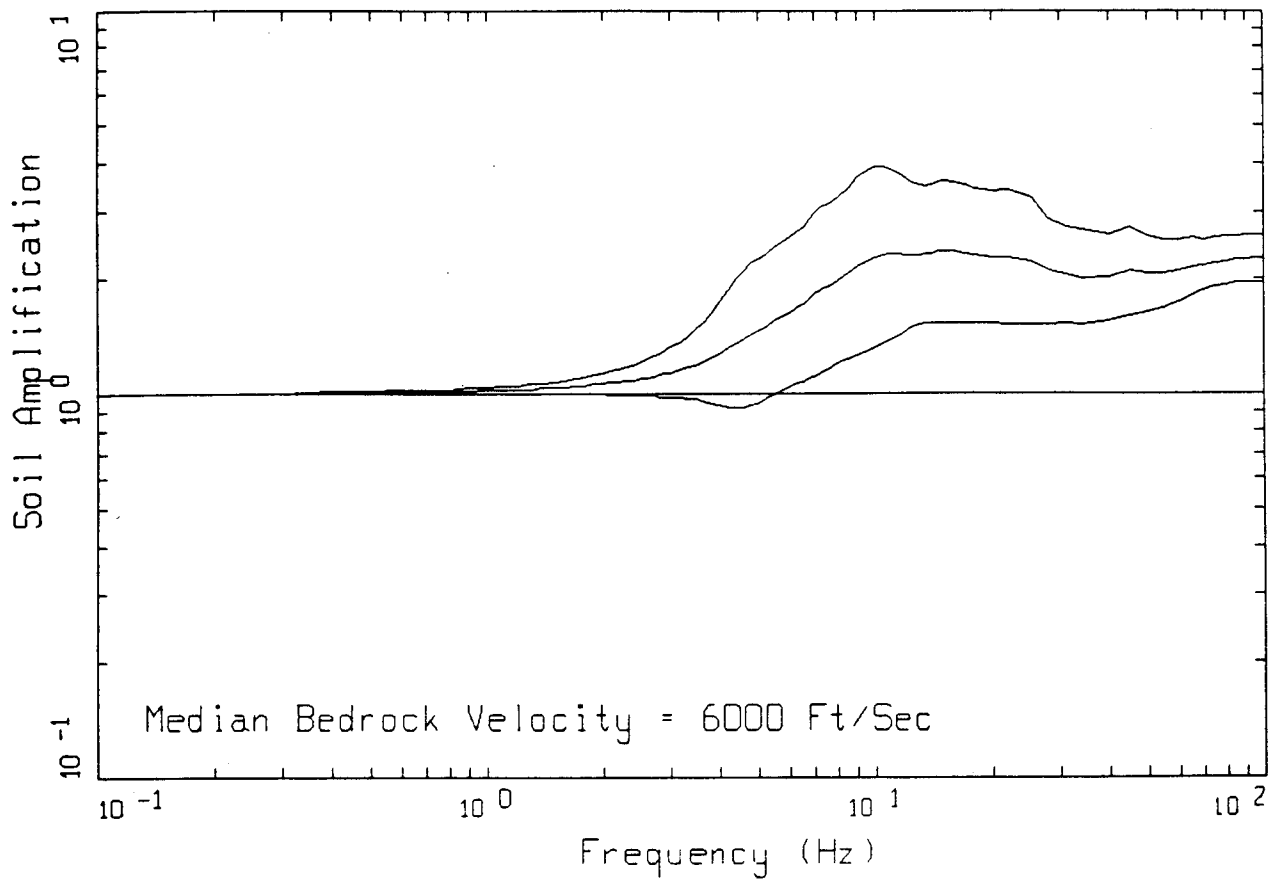


CATEGORY 1 (20 FT)

LEGEND	
—	CONTROL MOTION 0.05 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
—	CONTROL MOTION 0.05 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
—	CONTROL MOTION 0.05 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.

Figure Set 6.C-1

Category 1 (10–30 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

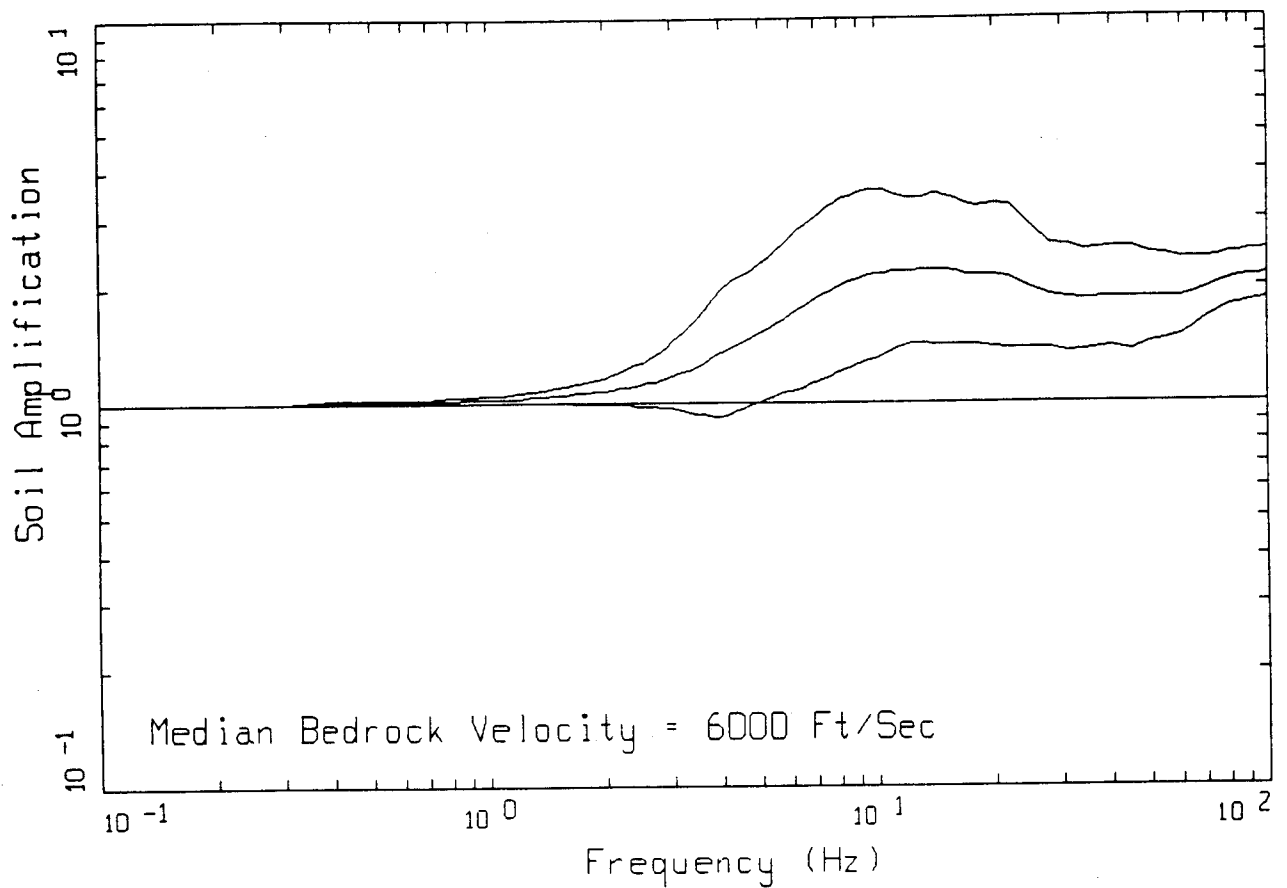


CATEGORY 1 (20 FT)

LEGEND	
—————	CONTROL MOTION 0.10 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
—————	CONTROL MOTION 0.10 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
—————	CONTROL MOTION 0.10 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.

Figure Set 6.C-1 (continued)

Category 1 (10–30 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

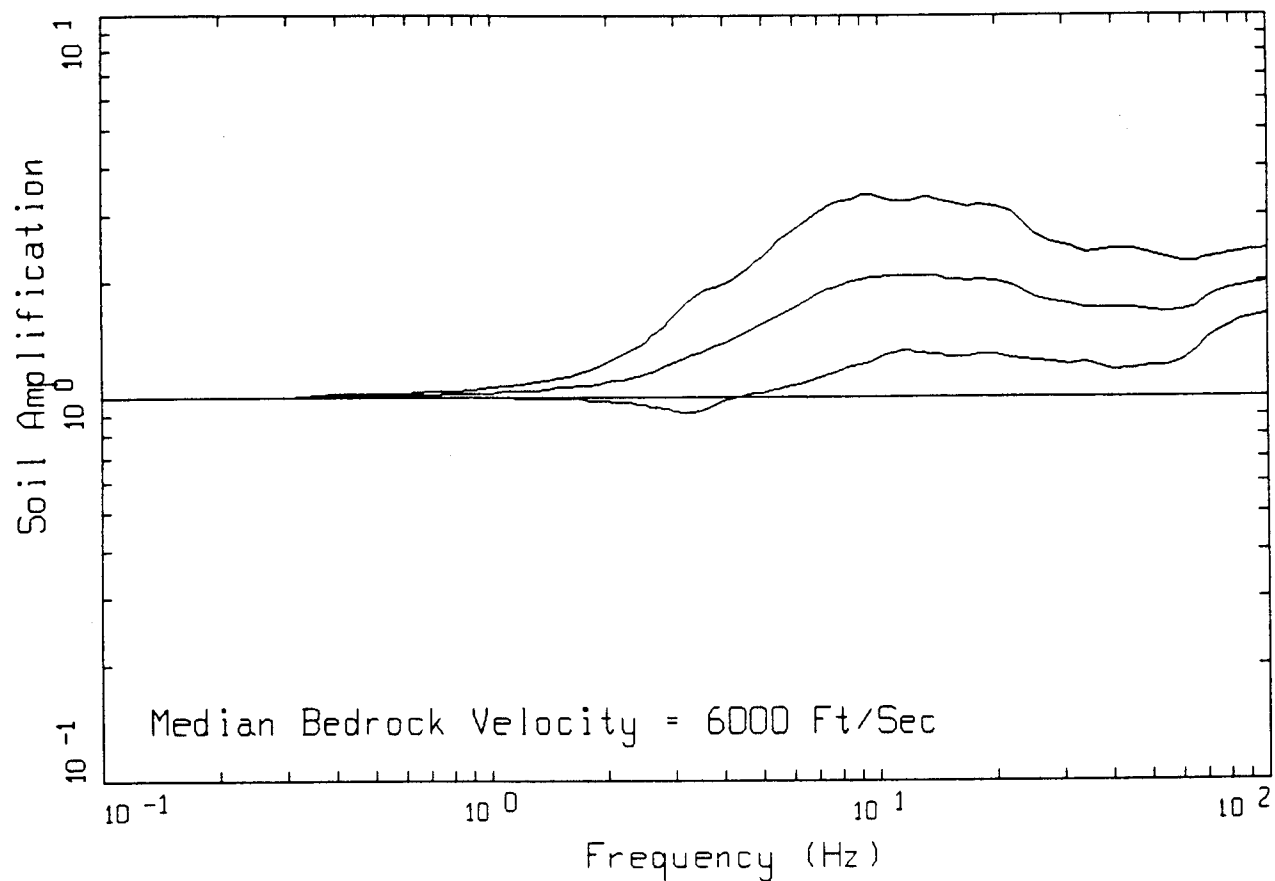


CATEGORY 1 (20 FT)

LEGEND	
————	CONTROL MOTION 0.20 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
————	CONTROL MOTION 0.20 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
————	CONTROL MOTION 0.20 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.

Figure Set 6.C-1 (continued)

Category 1 (10–30 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

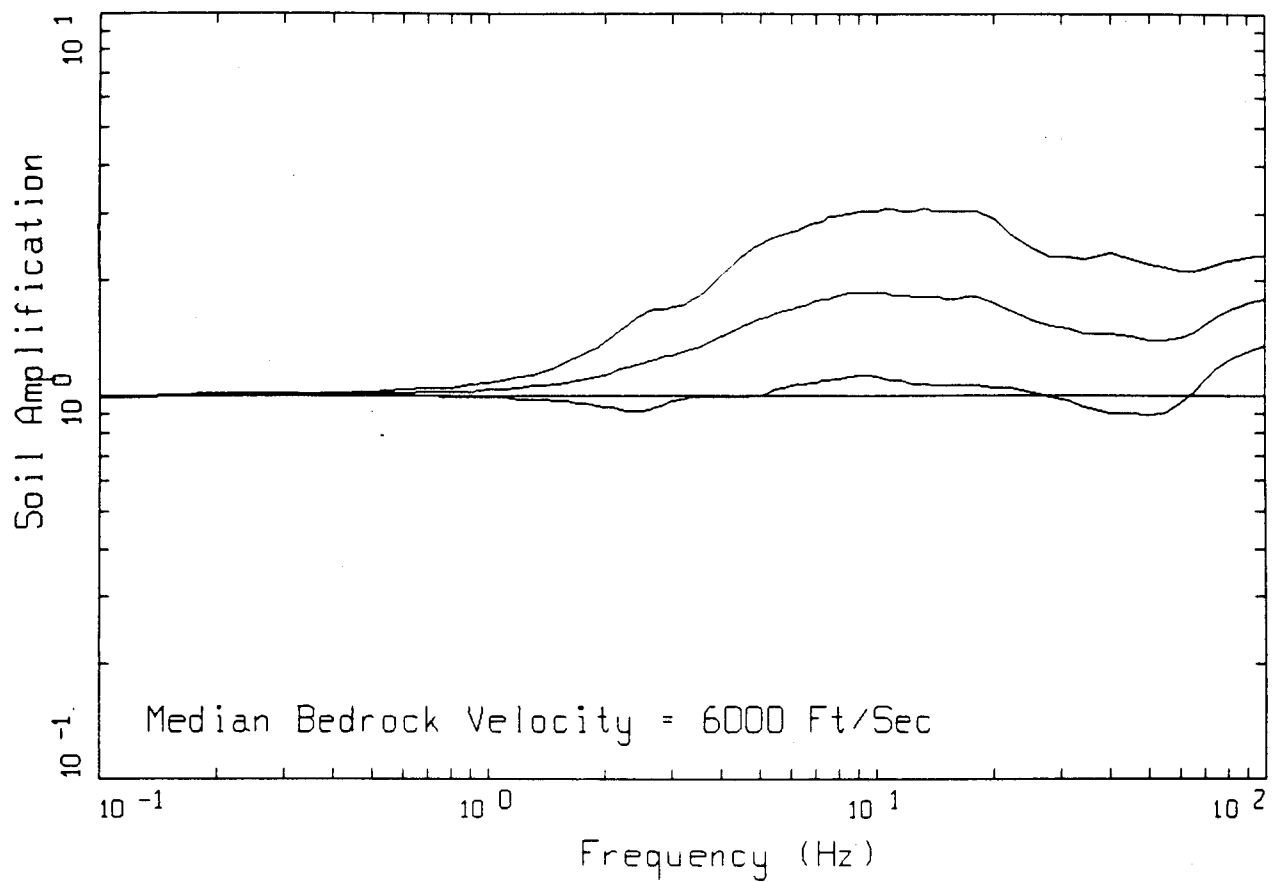


CATEGORY 1 (20 FT)

LEGEND	
—————	CONTROL MOTION 0.30 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
—————	CONTROL MOTION 0.30 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
—————	CONTROL MOTION 0.30 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.

Figure Set 6.C-1 (continued)

Category 1 (10–30 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

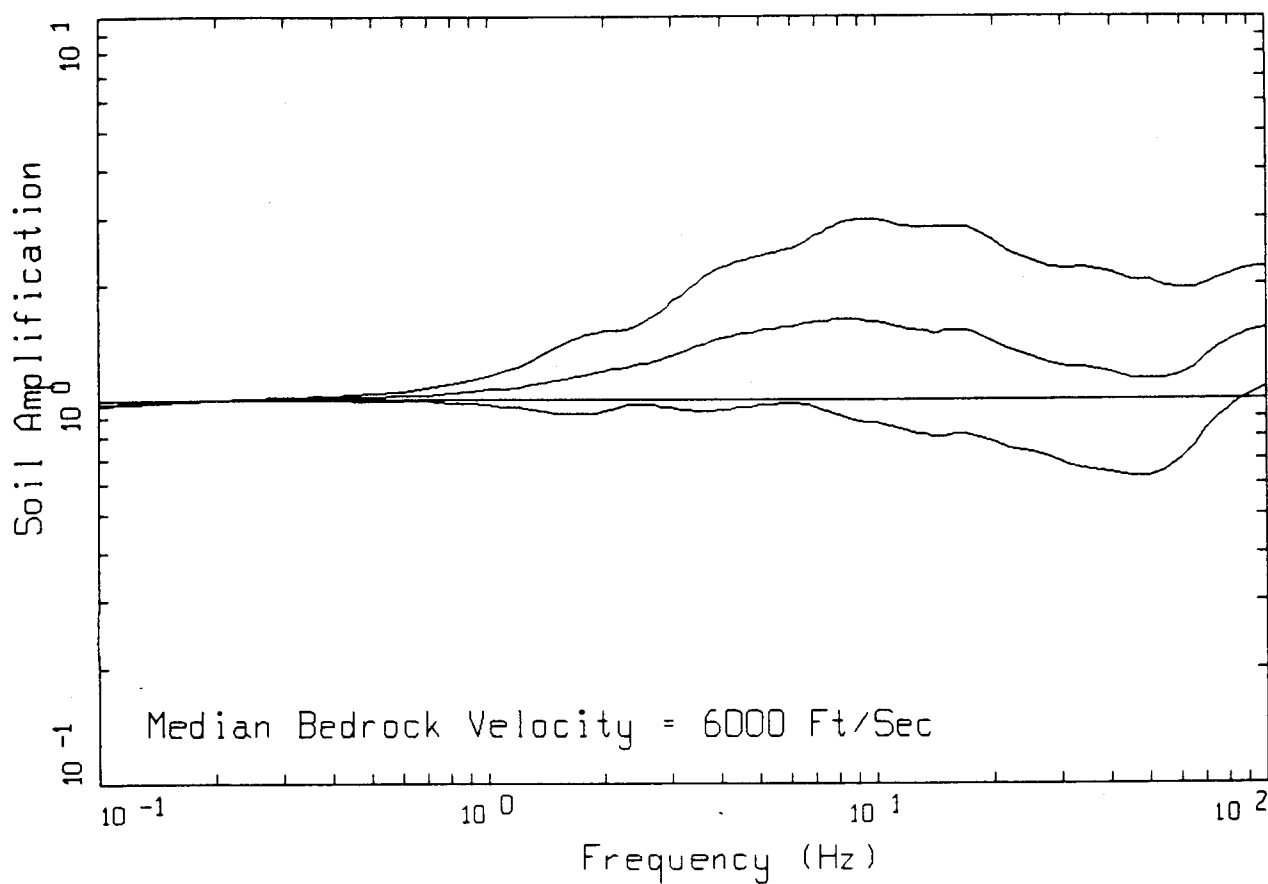


CATEGORY 1 (20 FT)

LEGEND	
—————	CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
—————	CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
—————	CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.

Figure Set 6.C-1 (continued)

Category 1 (10-30 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

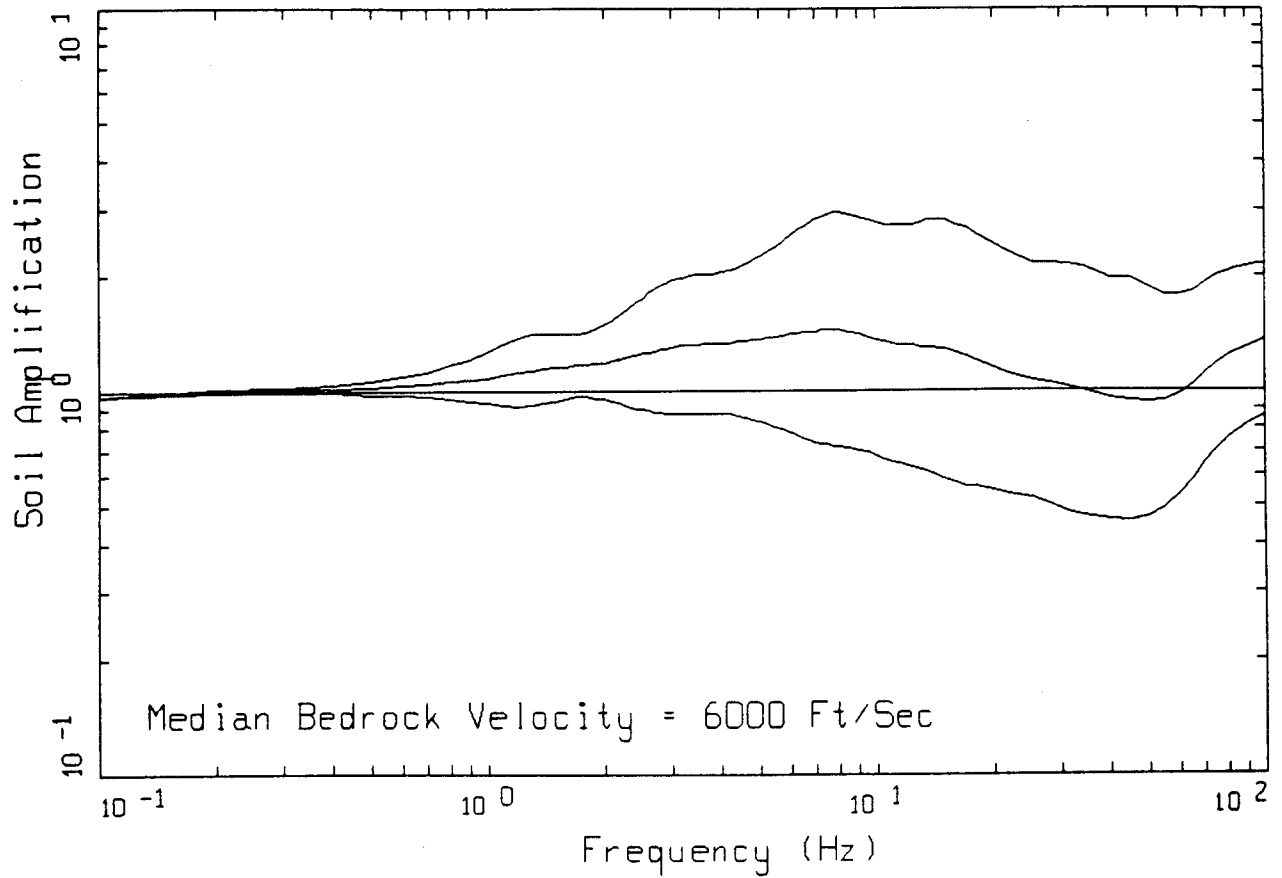


CATEGORY 1 (20 FT)

LEGEND	
————	CONTROL MOTION 0.75 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
————	CONTROL MOTION 0.75 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
————	CONTROL MOTION 0.75 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.

Figure Set 6.C-1 (continued)

Category 1 (10–30 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

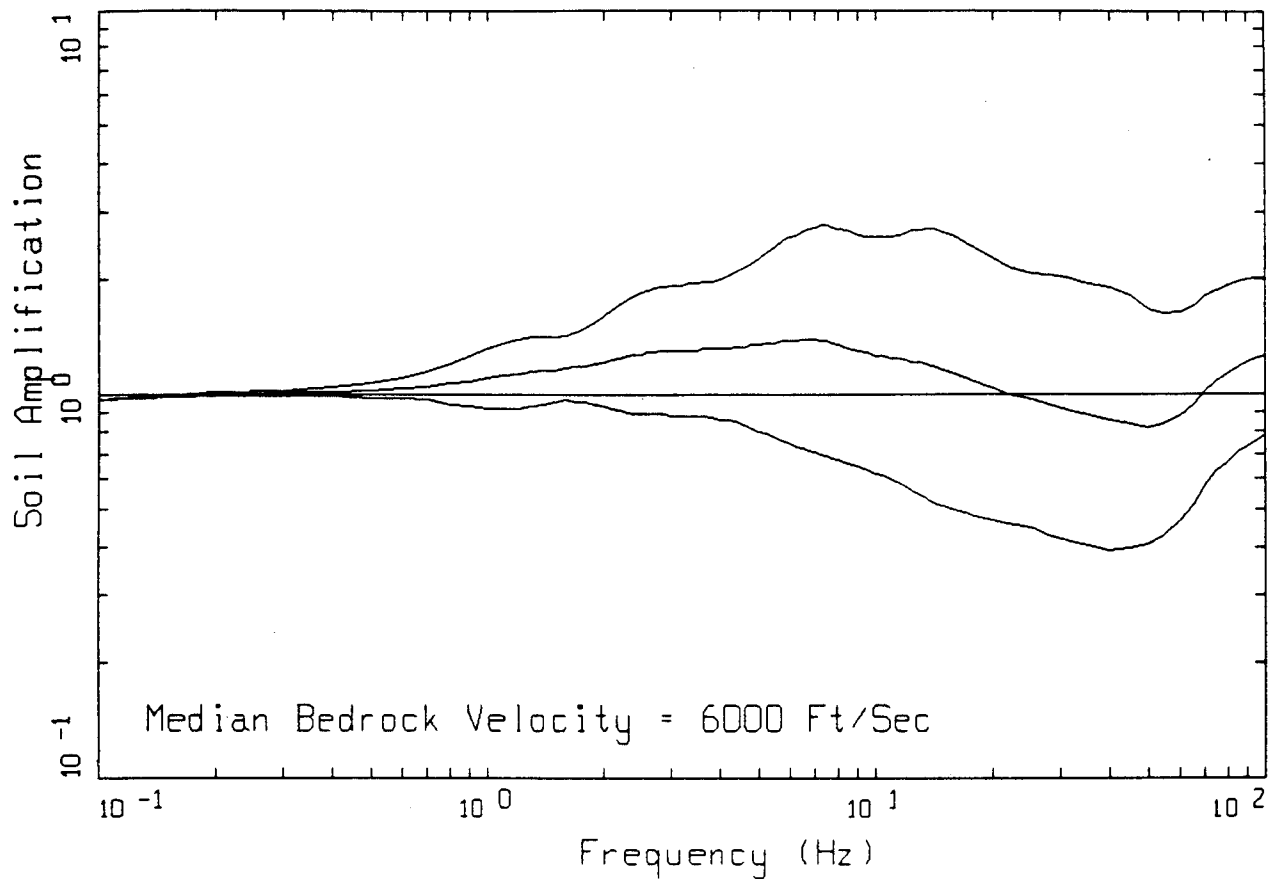


CATEGORY 1 (20 FT)

LEGEND	
—————	CONTROL MOTION 1.00 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
—————	CONTROL MOTION 1.00 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
—————	CONTROL MOTION 1.00 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.

Figure Set 6.C-1 (continued)

Category 1 (10–30 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

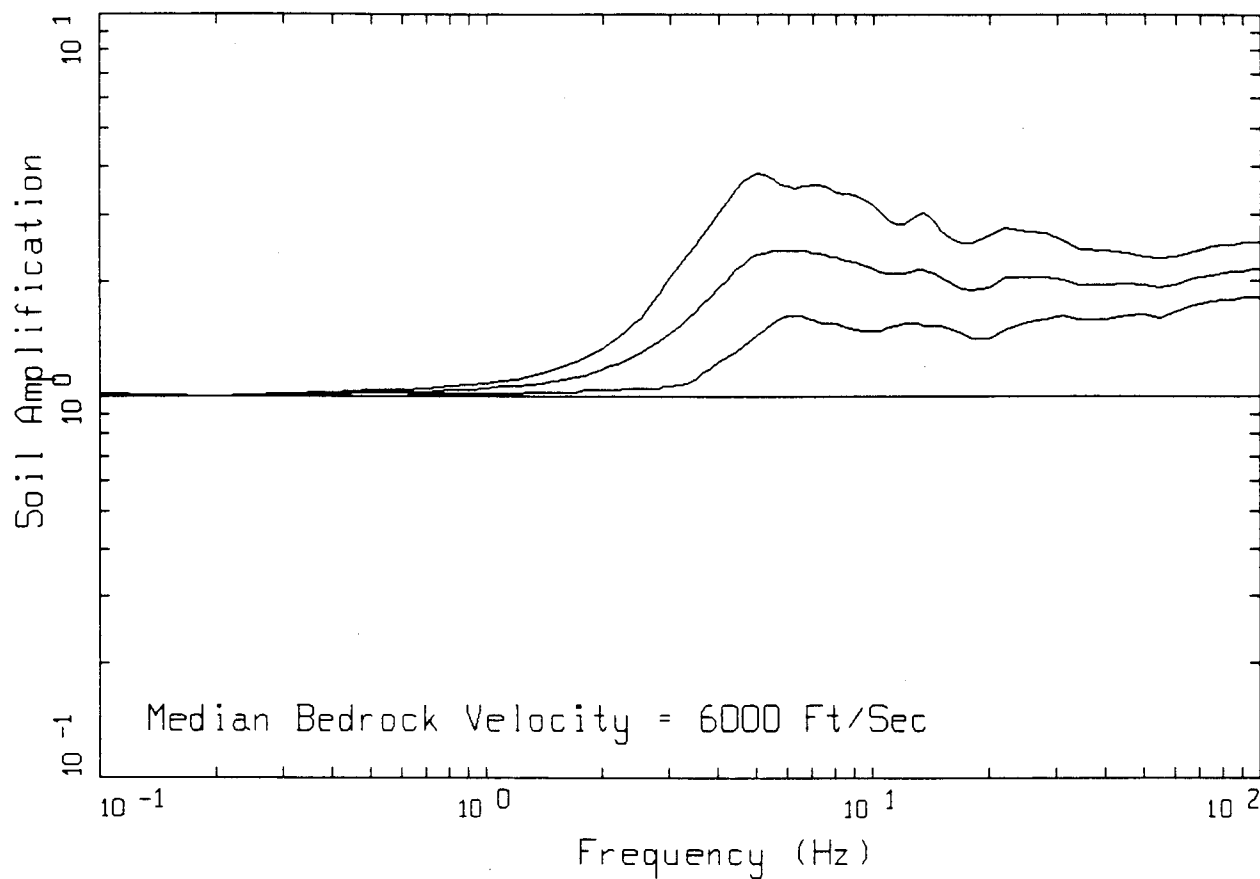


CATEGORY 1 (20 FT)

LEGEND	
————	CONTROL MOTION 1.25 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
————	CONTROL MOTION 1.25 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.
————	CONTROL MOTION 1.25 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 10 & 30 FT.

Figure Set 6.C-1 (continued)

Category 1 (10-30 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.



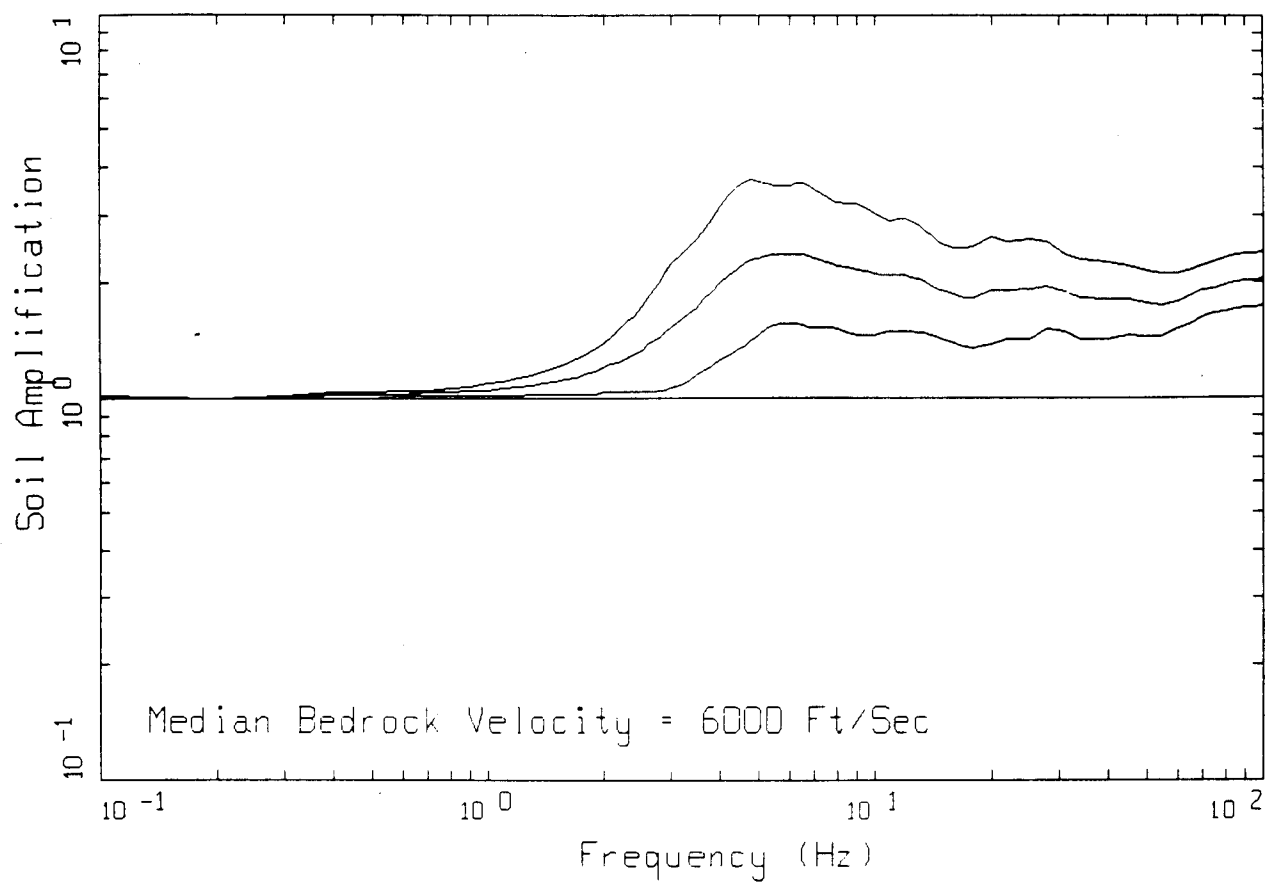
CATEGORY 2 (50 FT)

LEGEND

- CONTROL MOTION 0.05 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
- CONTROL MOTION 0.05 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
- CONTROL MOTION 0.05 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 31 & 80 FT.

Figure Set 6.C-2

Category 2 (31–80 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

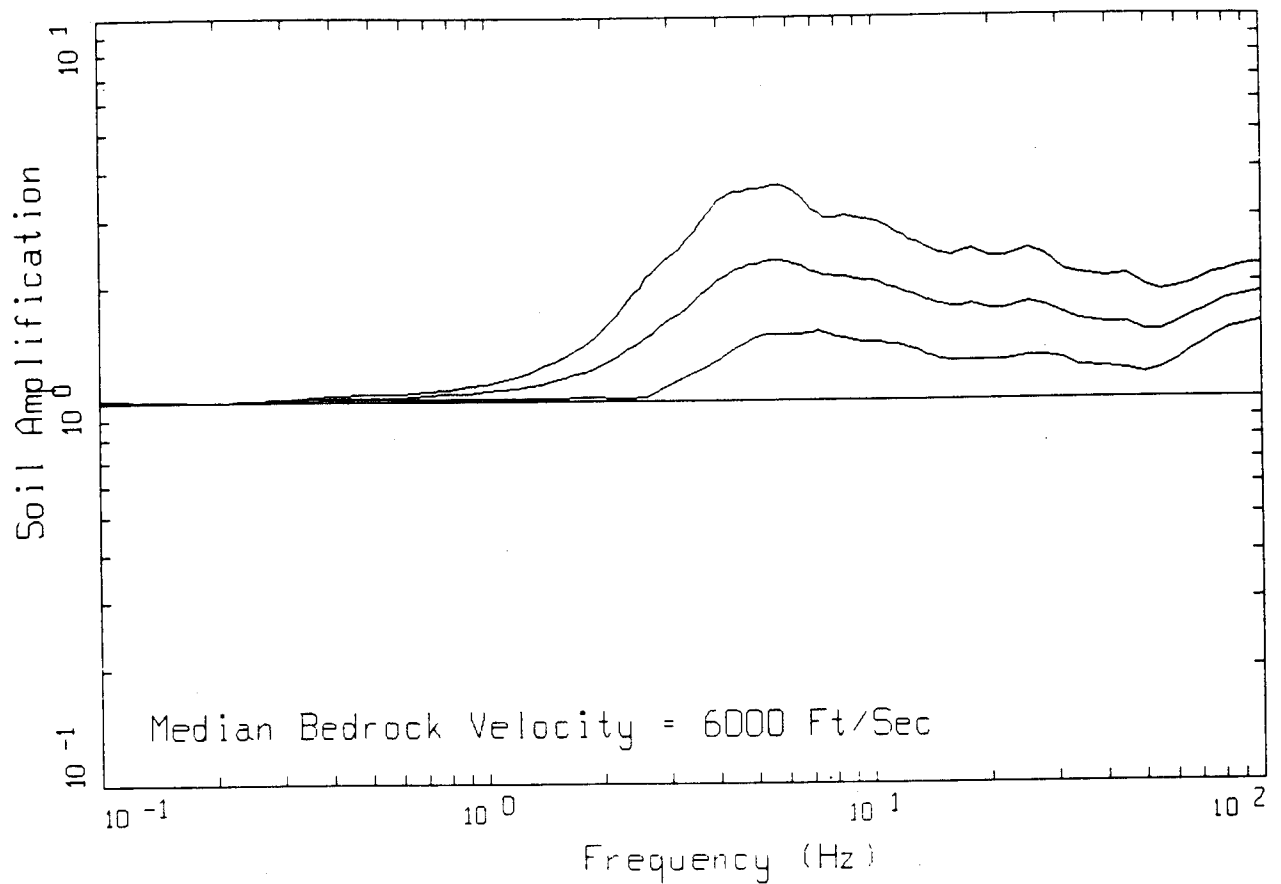


CATEGORY 2 (50 FT)

LEGEND	
—	CONTROL MOTION 0.10 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—	CONTROL MOTION 0.10 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—	CONTROL MOTION 0.10 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.

Figure Set 6.C-2 (continued)

Category 2 (31–80 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

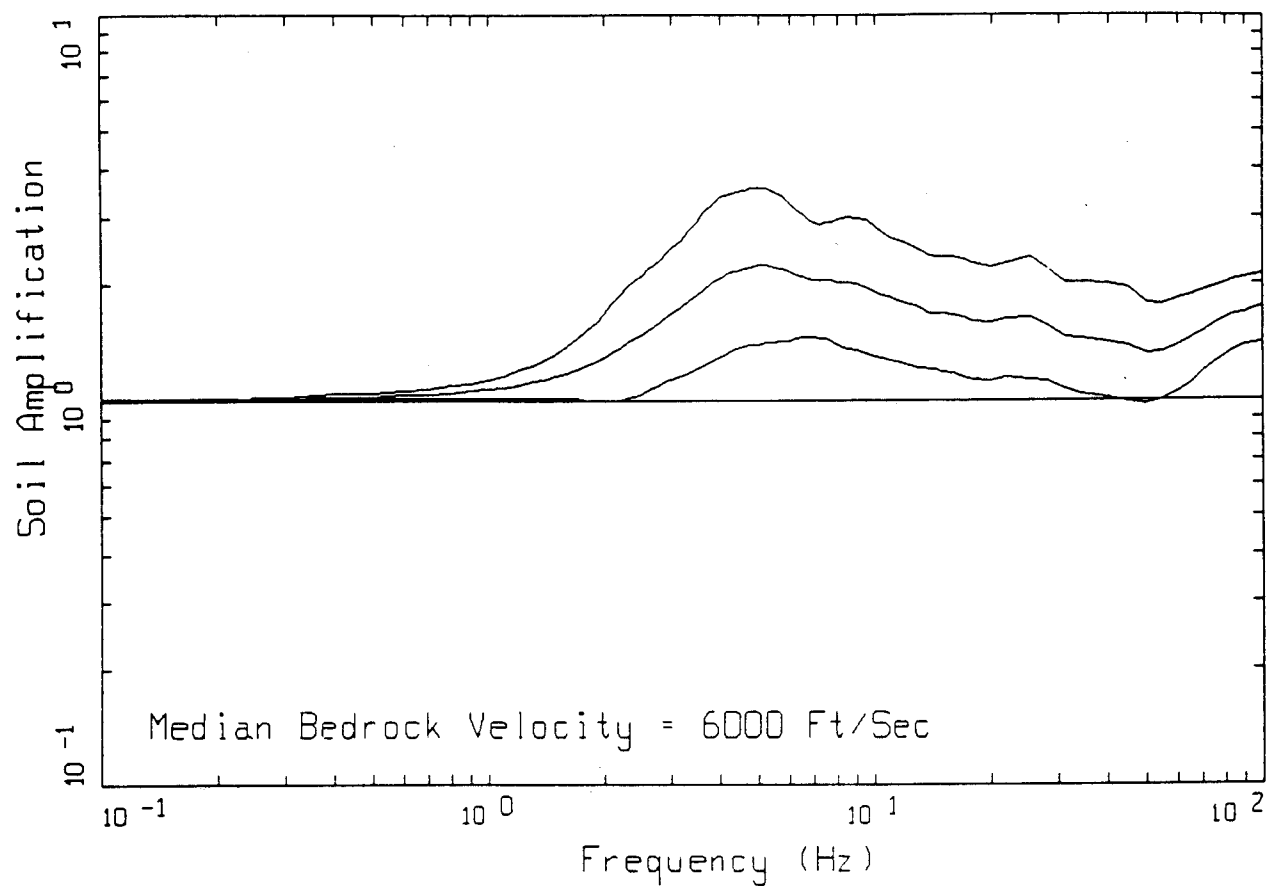


CATEGORY 2 (50 FT)

LEGEND	
—————	CONTROL MOTION 0.20 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—————	CONTROL MOTION 0.20 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—————	CONTROL MOTION 0.20 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.

Figure Set 6.C-2 (continued)

Category 2 (31–80 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

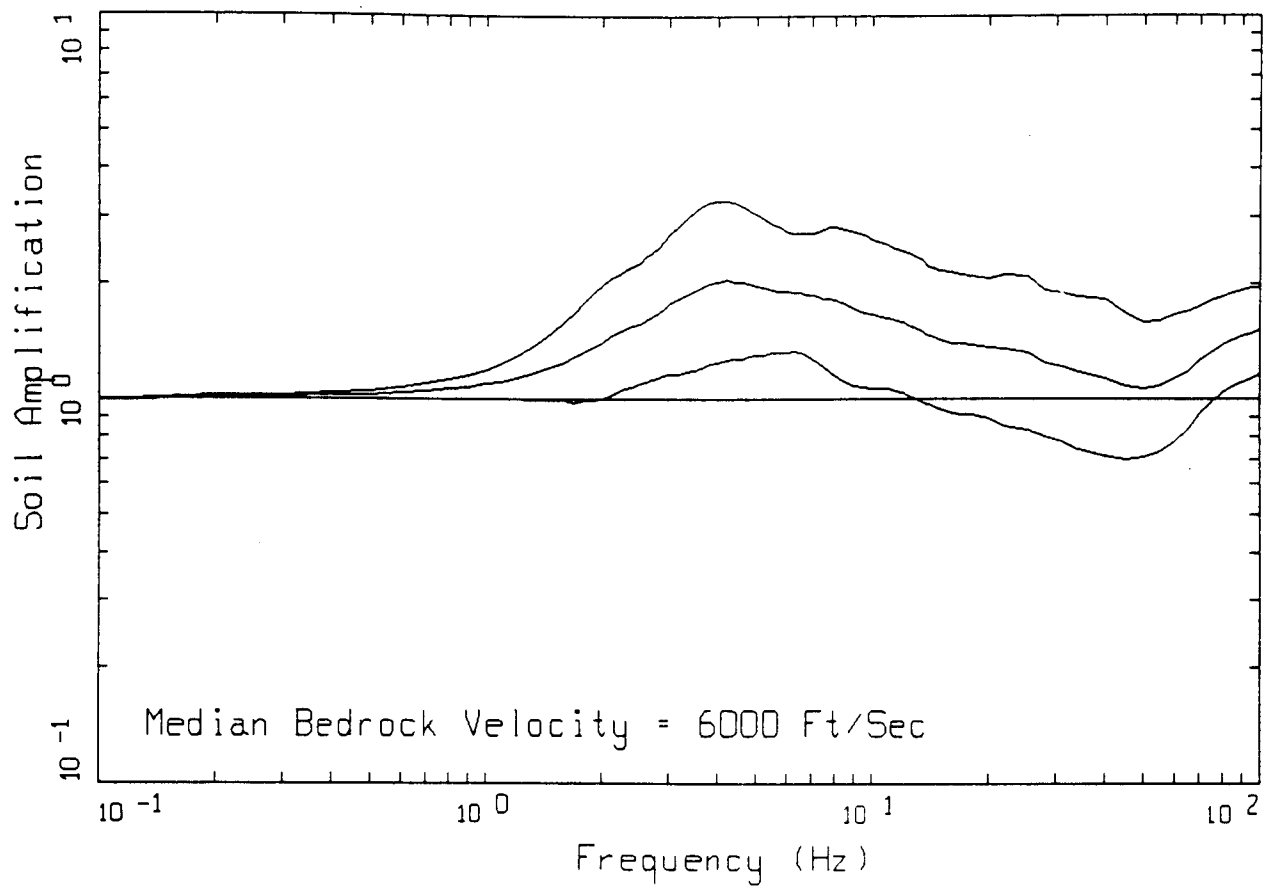


CATEGORY 2 (50 FT)

LEGEND	
—	CONTROL MOTION 0.30 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—	CONTROL MOTION 0.30 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—	CONTROL MOTION 0.30 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.

Figure Set 6.C-2 (continued)

Category 2 (31–80 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

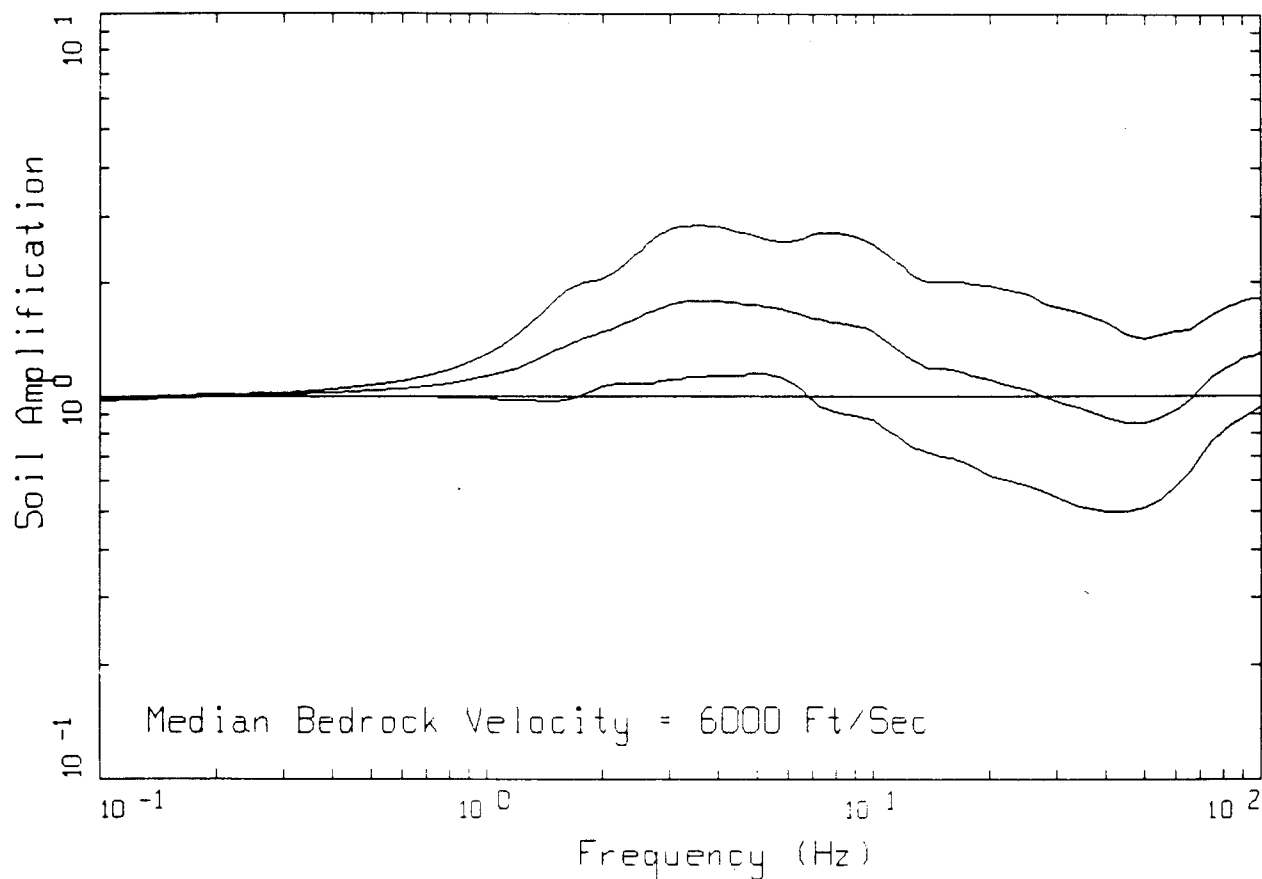


CATEGORY 2 (50 FT)

LEGEND	
—————	CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—————	CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—————	CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.

Figure Set 6.C-2 (continued)

Category 2 (31–80 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

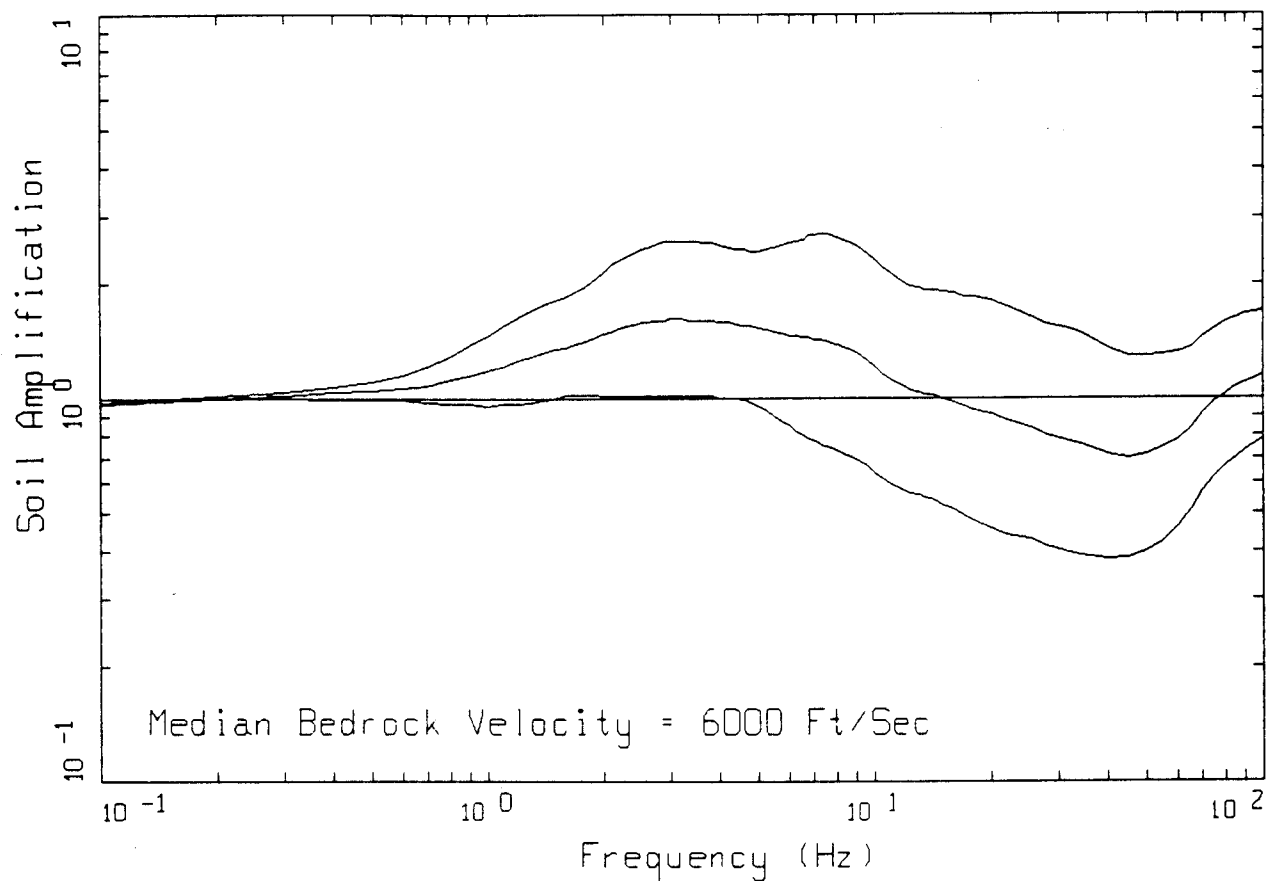


CATEGORY 2 (50 FT)

LEGEND	
—	CONTROL MOTION 0.75 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—	CONTROL MOTION 0.75 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—	CONTROL MOTION 0.75 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.

Figure Set 6.C-2 (continued)

Category 2 (31–80 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

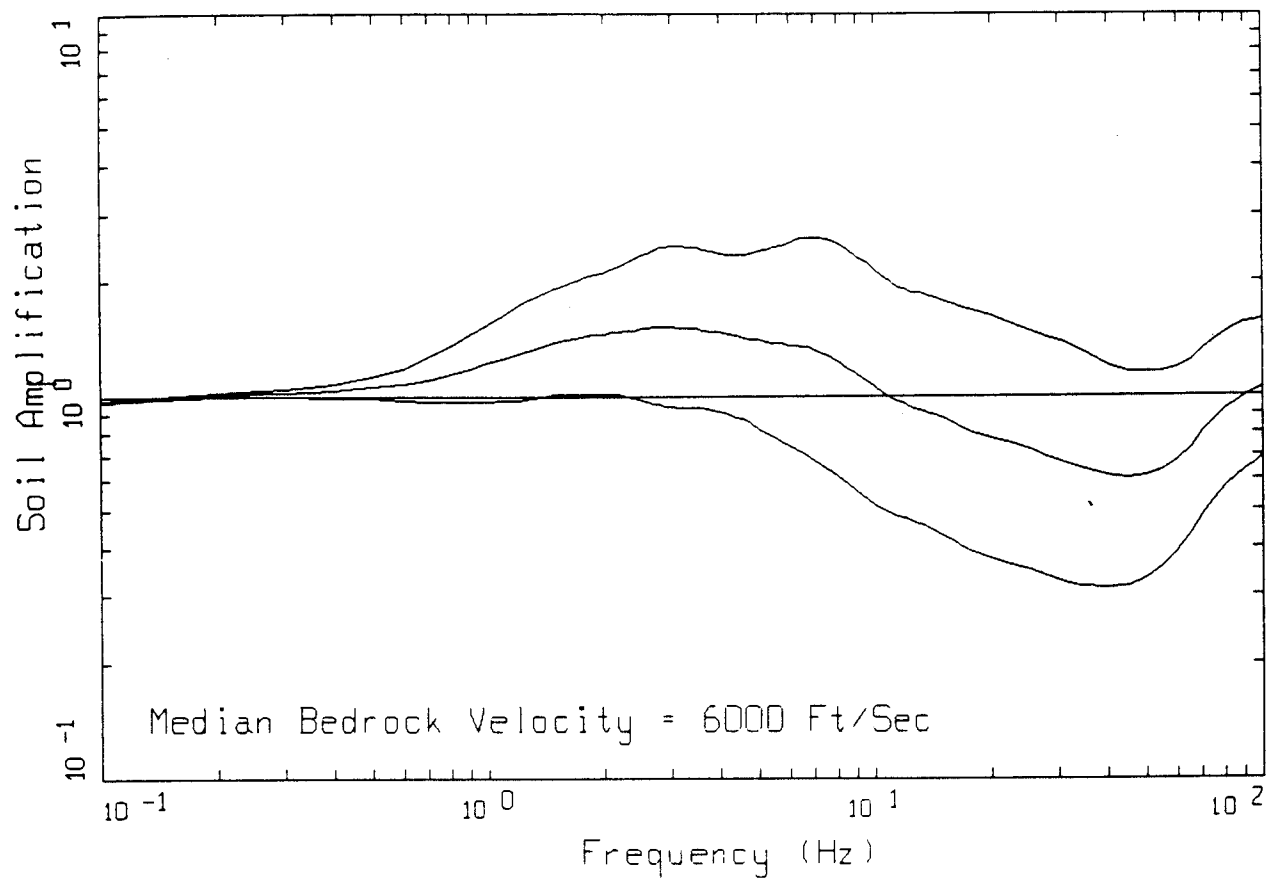


CATEGORY 2 (50 FT)

LEGEND	
—————	CONTROL MOTION 1.00 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—————	CONTROL MOTION 1.00 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—————	CONTROL MOTION 1.00 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.

Figure Set 6.C-2 (continued)

Category 2 (31–80 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

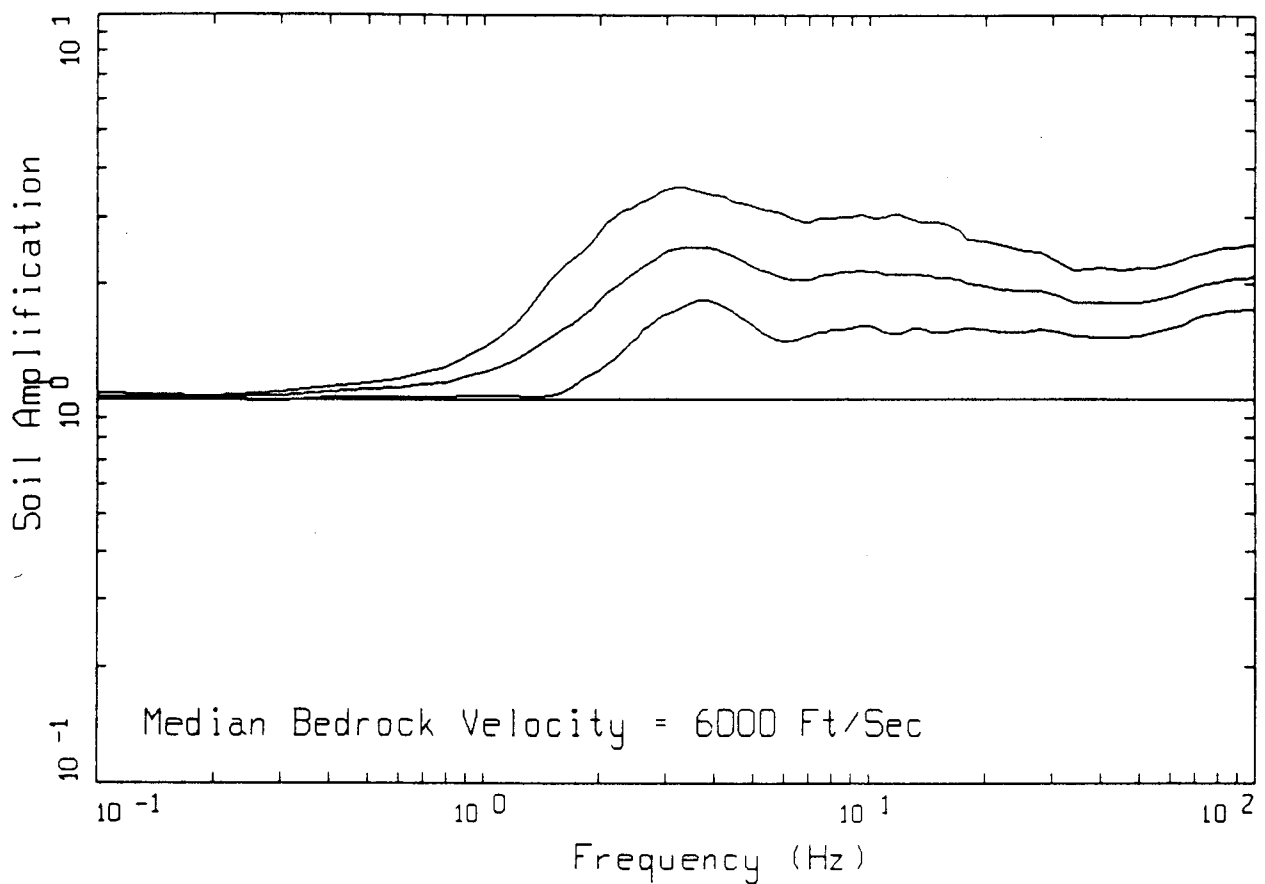


CATEGORY 2 (50 FT)

LEGEND	
—	CONTROL MOTION 1.25 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—	CONTROL MOTION 1.25 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.
—	CONTROL MOTION 1.25 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT.

Figure Set 6.C-2 (continued)

Category 2 (31–80 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

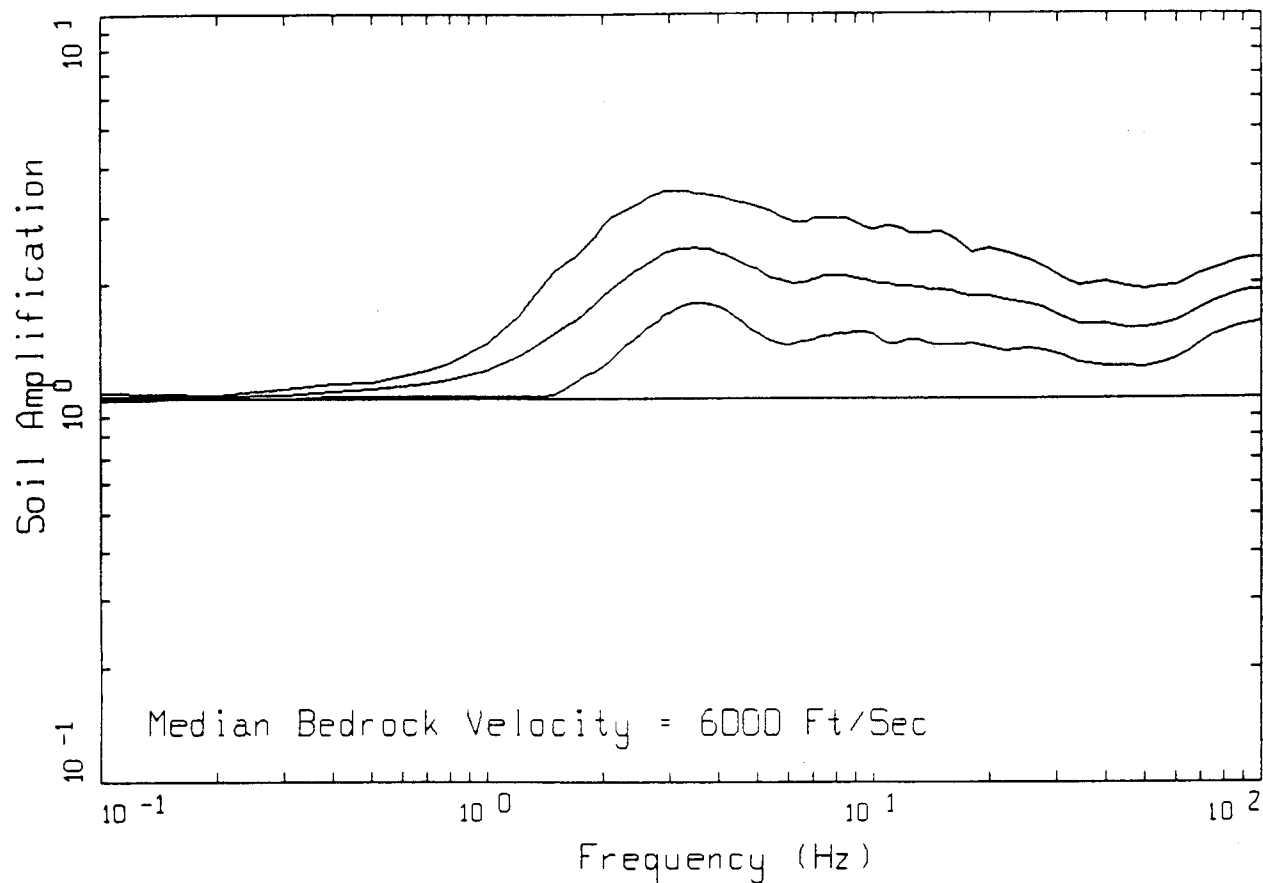


CATEGORY 3 (120 FT)

LEGEND	
—————	CONTROL MOTION 0.05 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—————	CONTROL MOTION 0.05 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—————	CONTROL MOTION 0.05 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.

Figure Set 6.C-3

Category 3 (81–180 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

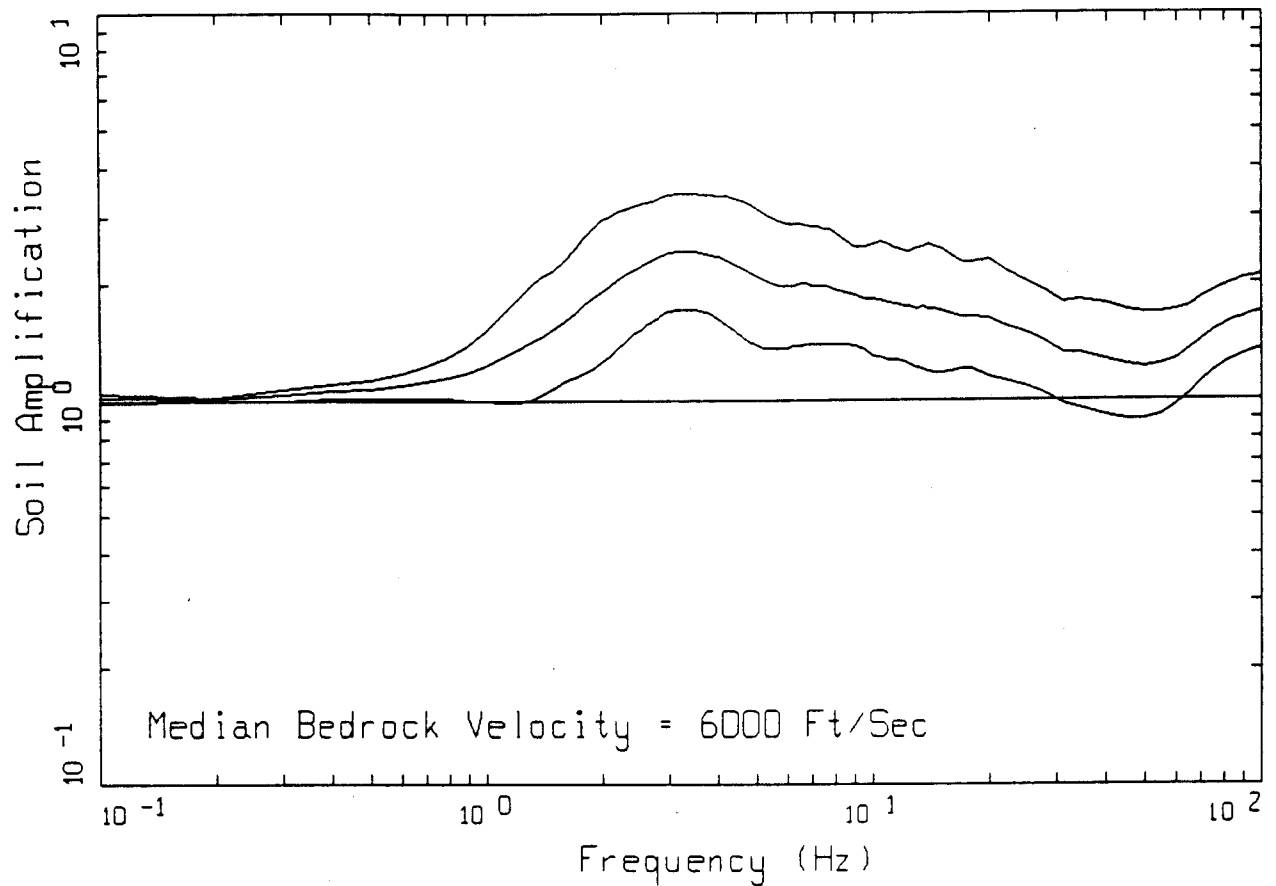


CATEGORY 3 (120 FT)

LEGEND	
—	CONTROL MOTION 0.10 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—	CONTROL MOTION 0.10 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—	CONTROL MOTION 0.10 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.

Figure Set 6.C-3 (continued)

Category 3 (81–180 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

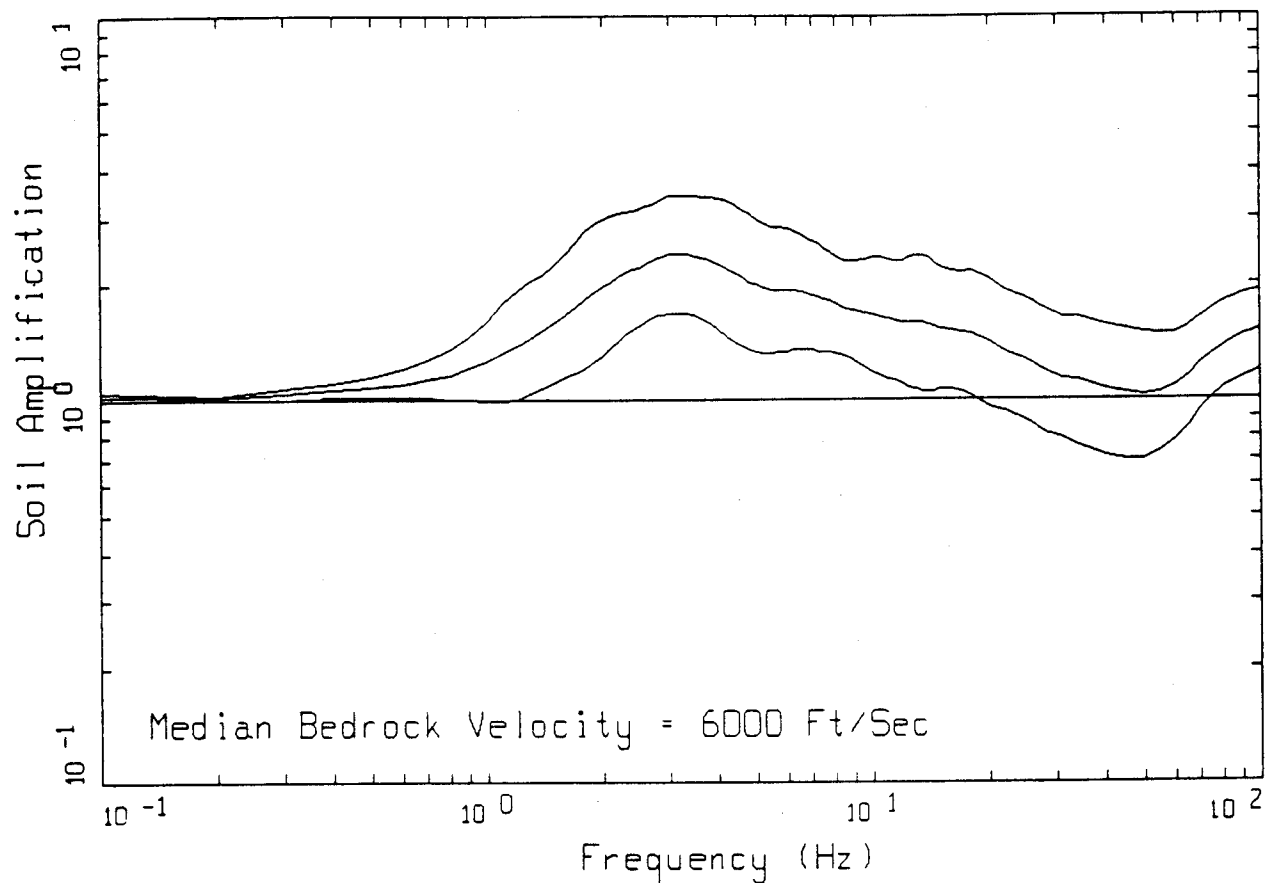


CATEGORY 3 (120 FT)

LEGEND	
————	CONTROL MOTION 0.20 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
————	CONTROL MOTION 0.20 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
————	CONTROL MOTION 0.20 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.

Figure Set 6.C-3 (continued)

Category 3 (81–180 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.



CATEGORY 3 (120 FT)

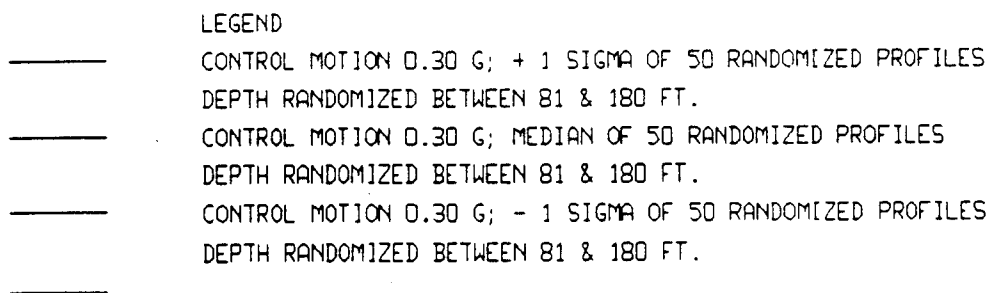
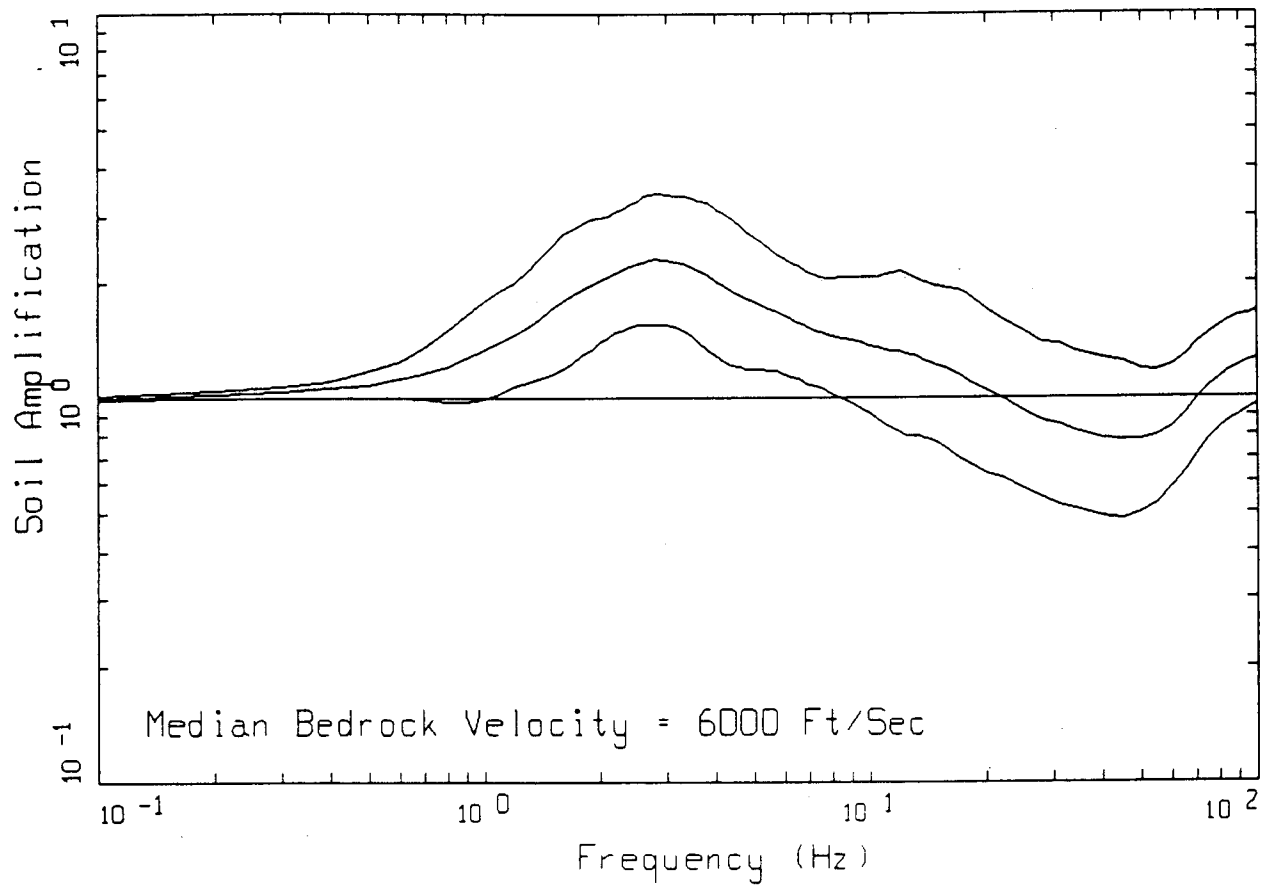


Figure Set 6.C-3 (continued)

Category 3 (81–180 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

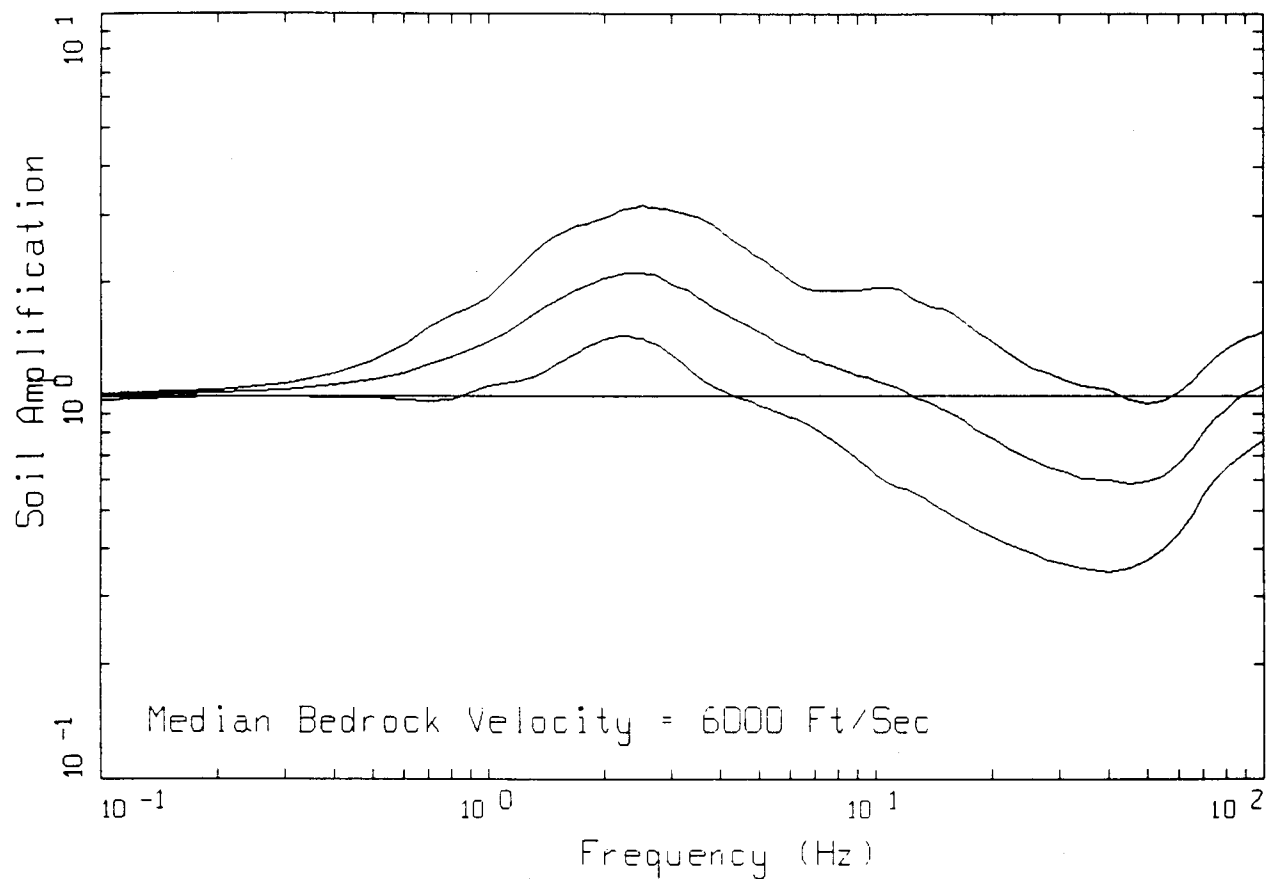


CATEGORY 3 (120 FT)

LEGEND	
—————	CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—————	CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—————	CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.

Figure Set 6.C-3 (continued)

Category 3 (81–180 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

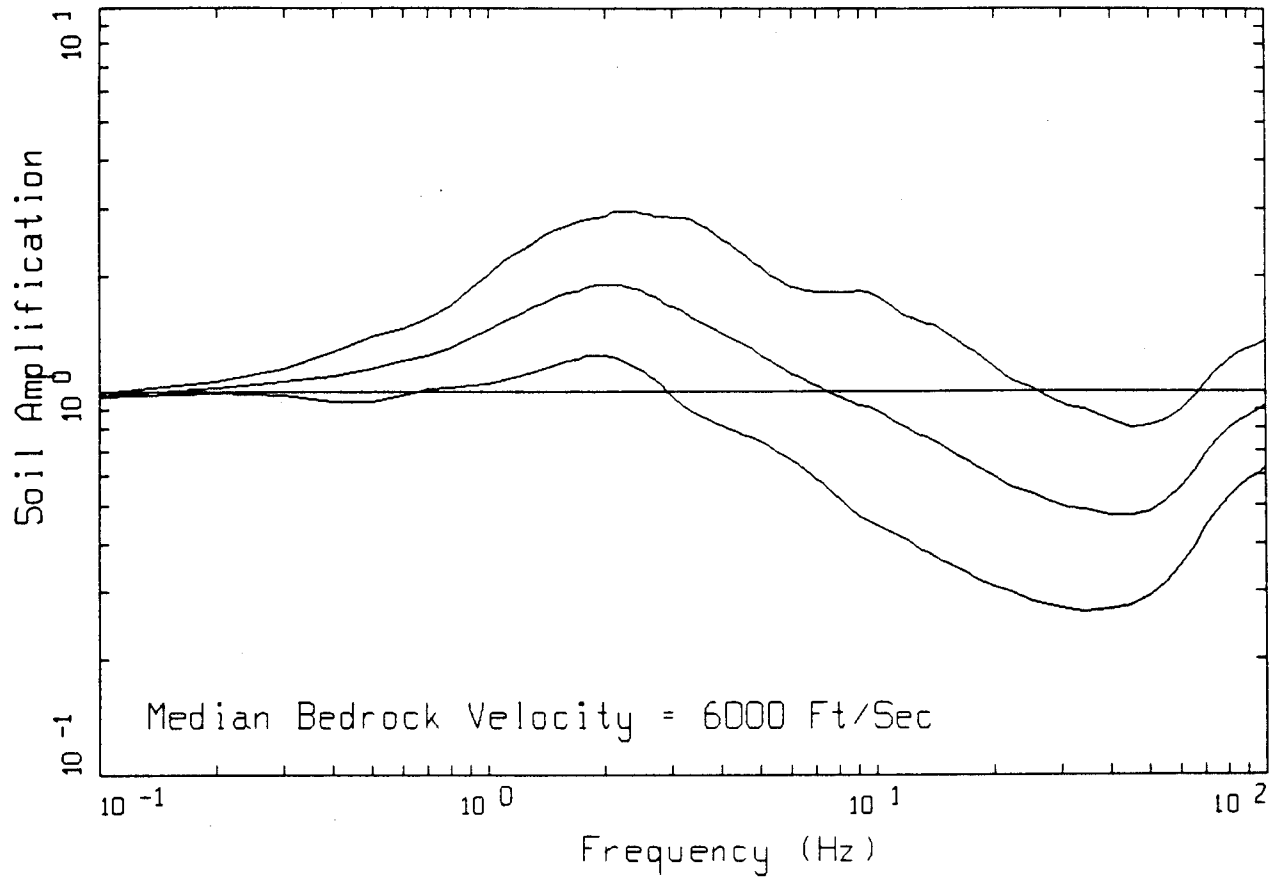


CATEGORY 3 (120 FT)

LEGEND	
—————	CONTROL MOTION 0.75 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—————	CONTROL MOTION 0.75 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—————	CONTROL MOTION 0.75 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.

Figure Set 6.C-3 (continued)

Category 3 (81–180 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.



CATEGORY 3 (120 FT)

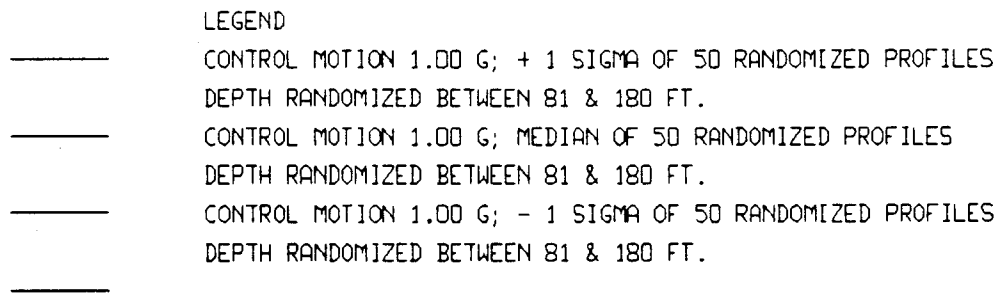
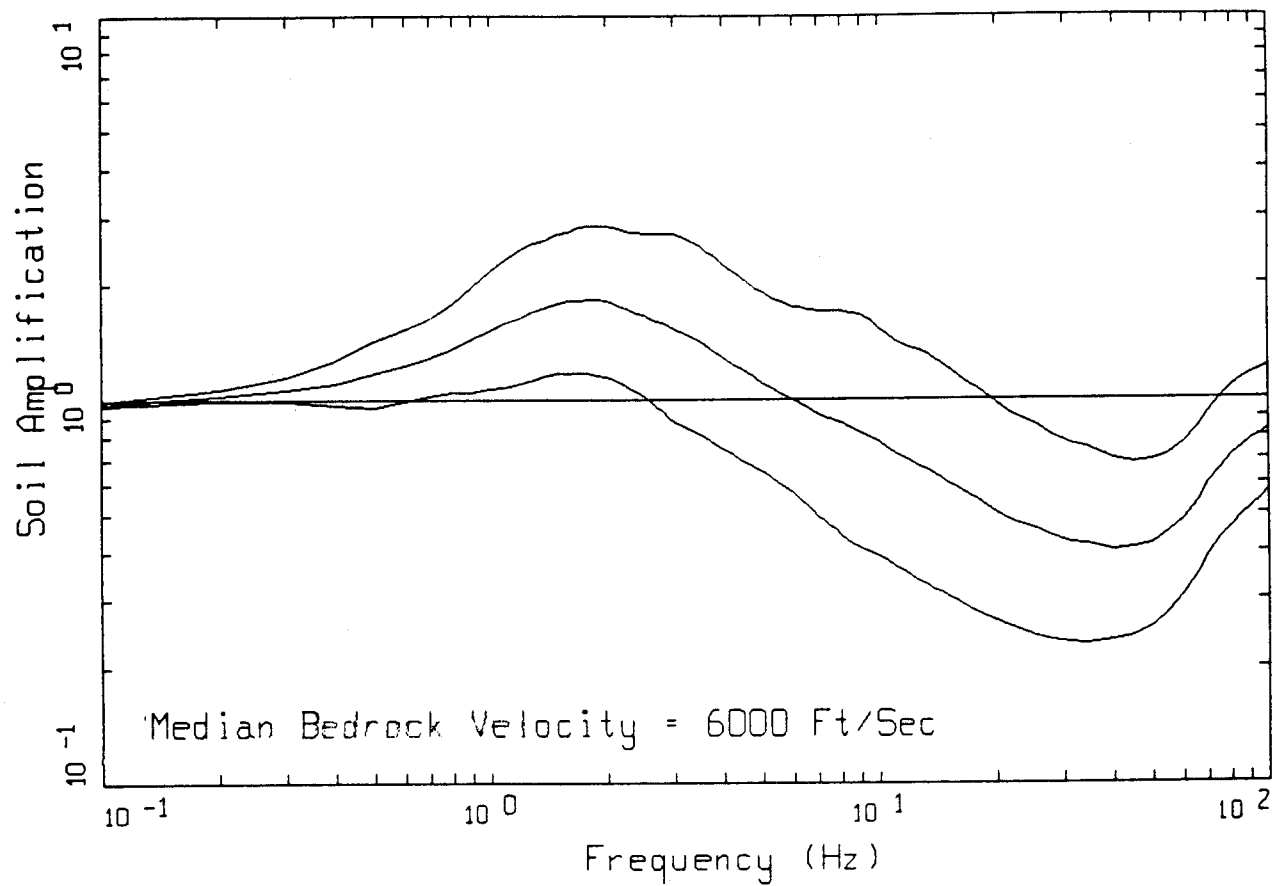


Figure Set 6.C-3 (continued)

Category 3 (81-180 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

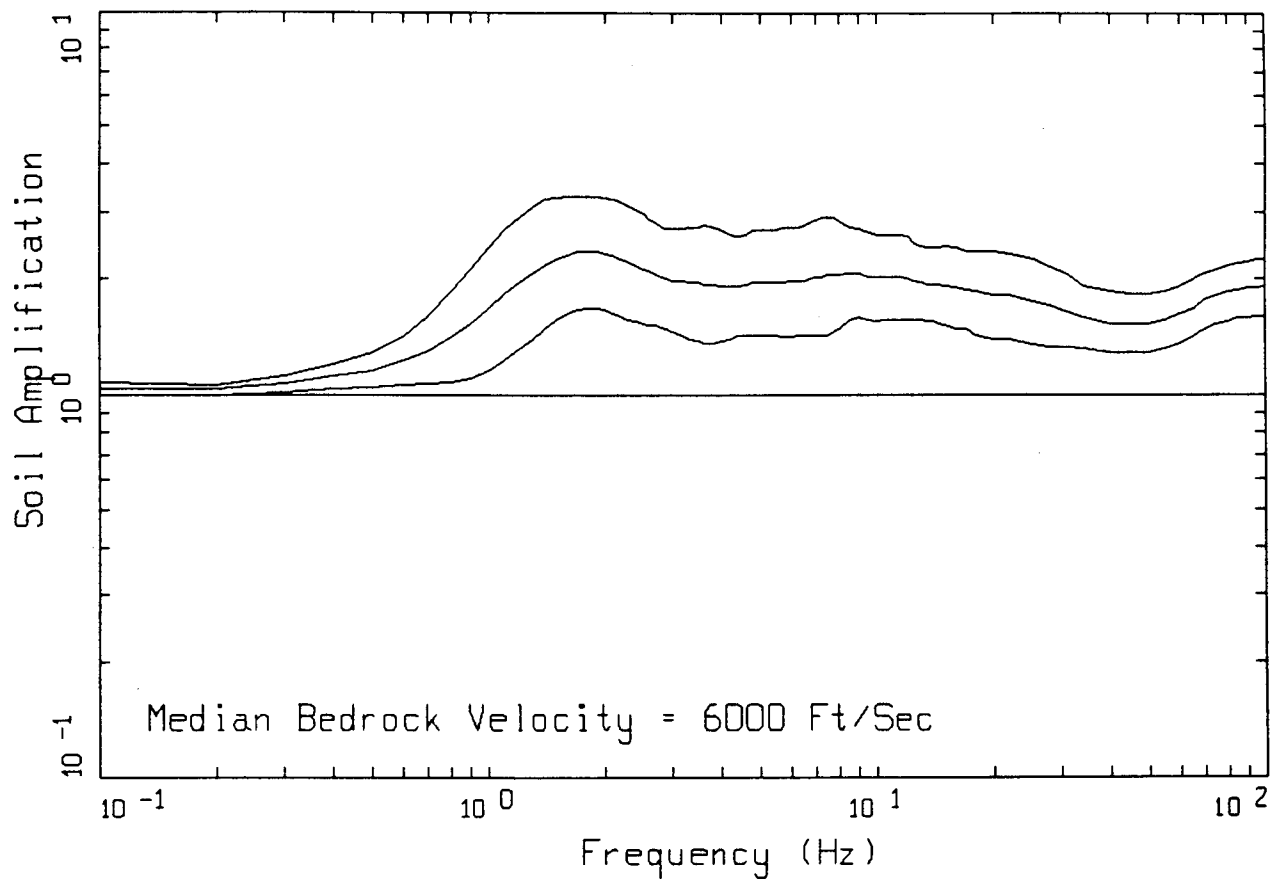


CATEGORY 3 (120 FT)

LEGEND	
—	CONTROL MOTION 1.25 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—	CONTROL MOTION 1.25 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—	CONTROL MOTION 1.25 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.

Figure Set 6.C-3 (continued)

Category 3 (81–180 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

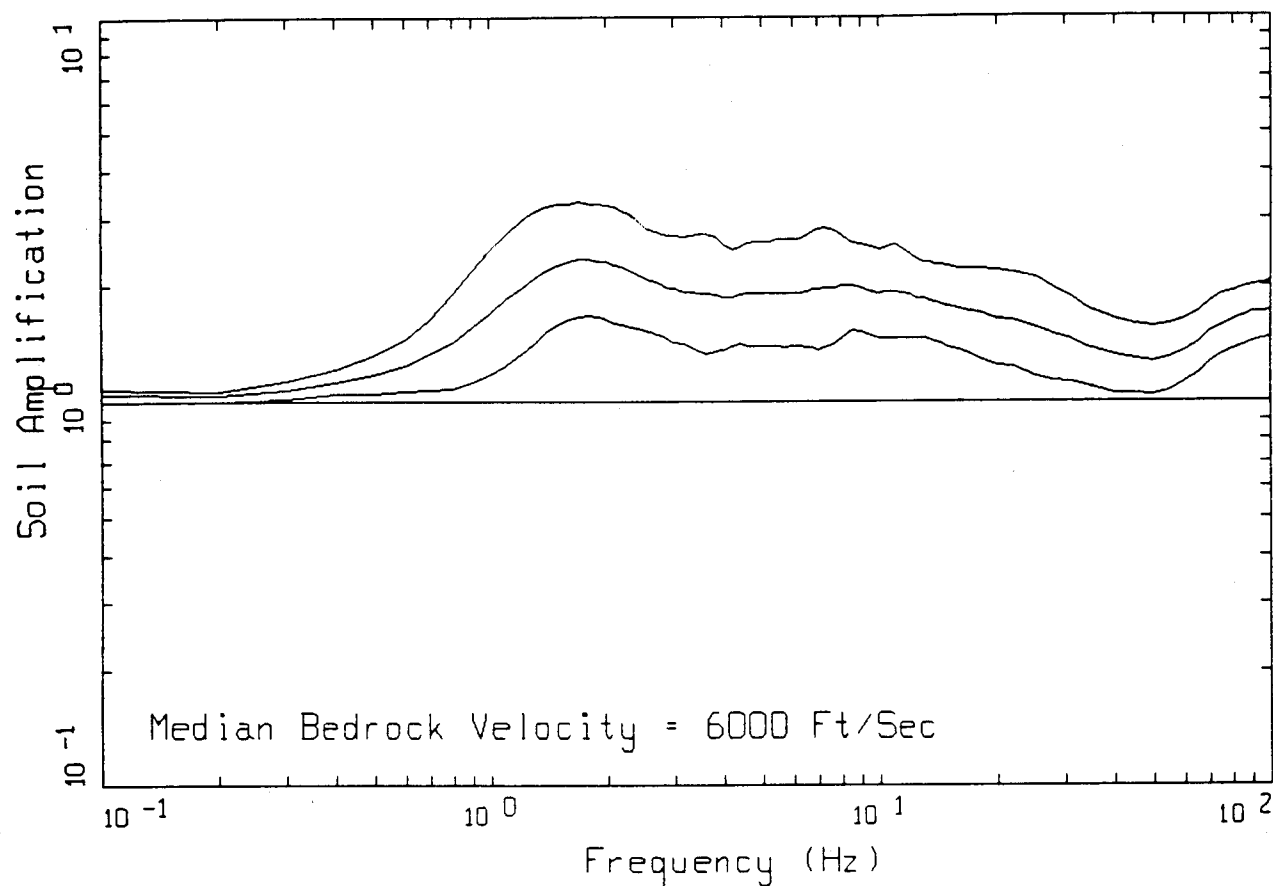


CATEGORY 4 (250 FT)

LEGEND	
—————	CONTROL MOTION 0.05 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
—————	CONTROL MOTION 0.05 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
—————	CONTROL MOTION 0.05 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.

Figure Set 6.C-4

Category 4 (181–400 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

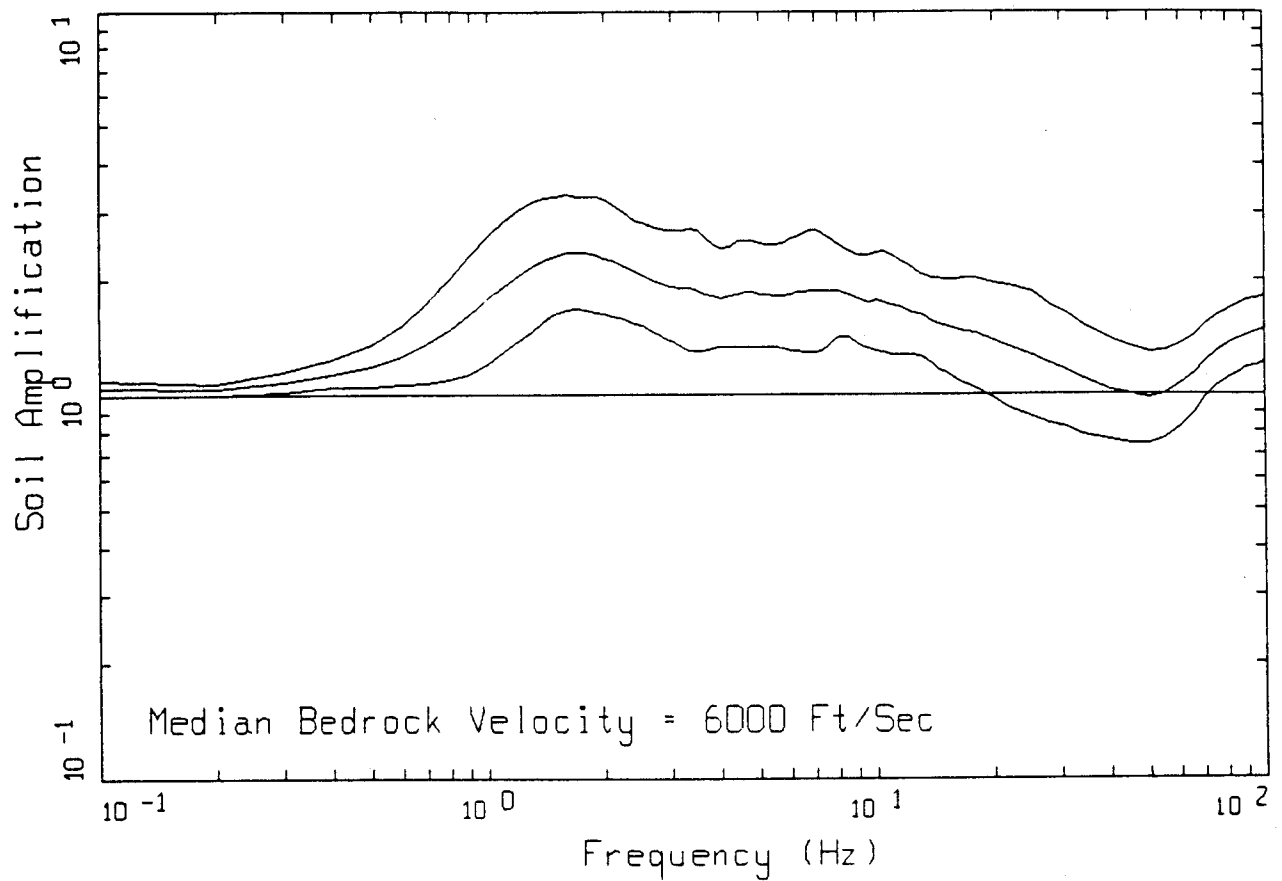


CATEGORY 4 (250 FT)

LEGEND	
————	CONTROL MOTION 0.10 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
————	CONTROL MOTION 0.10 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
————	CONTROL MOTION 0.10 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.

Figure Set 6.C-4 (continued)

Category 4 (181–400 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.



CATEGORY 4 (250 FT)

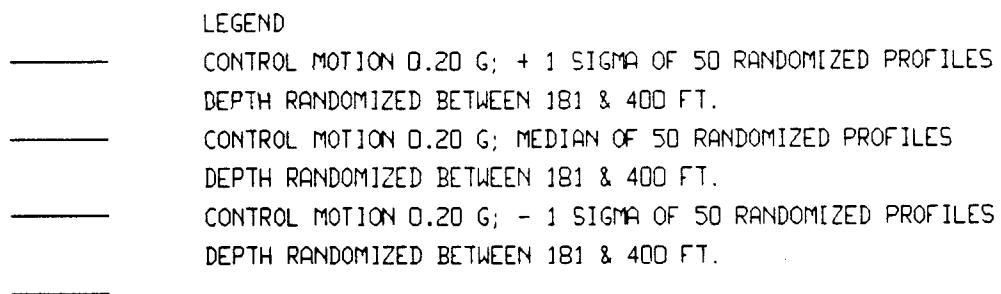
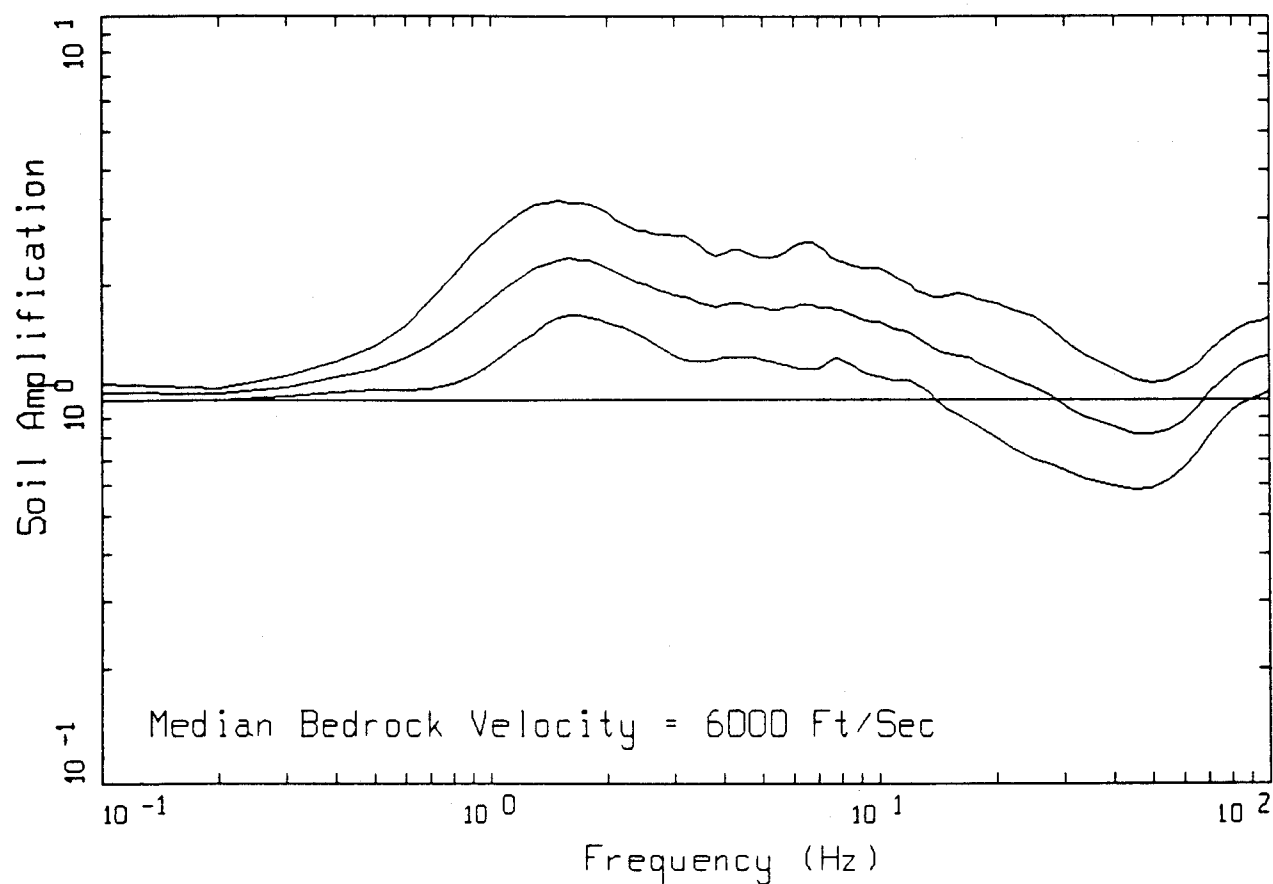


Figure Set 6.C-4 (continued)

Category 4 (181–400 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

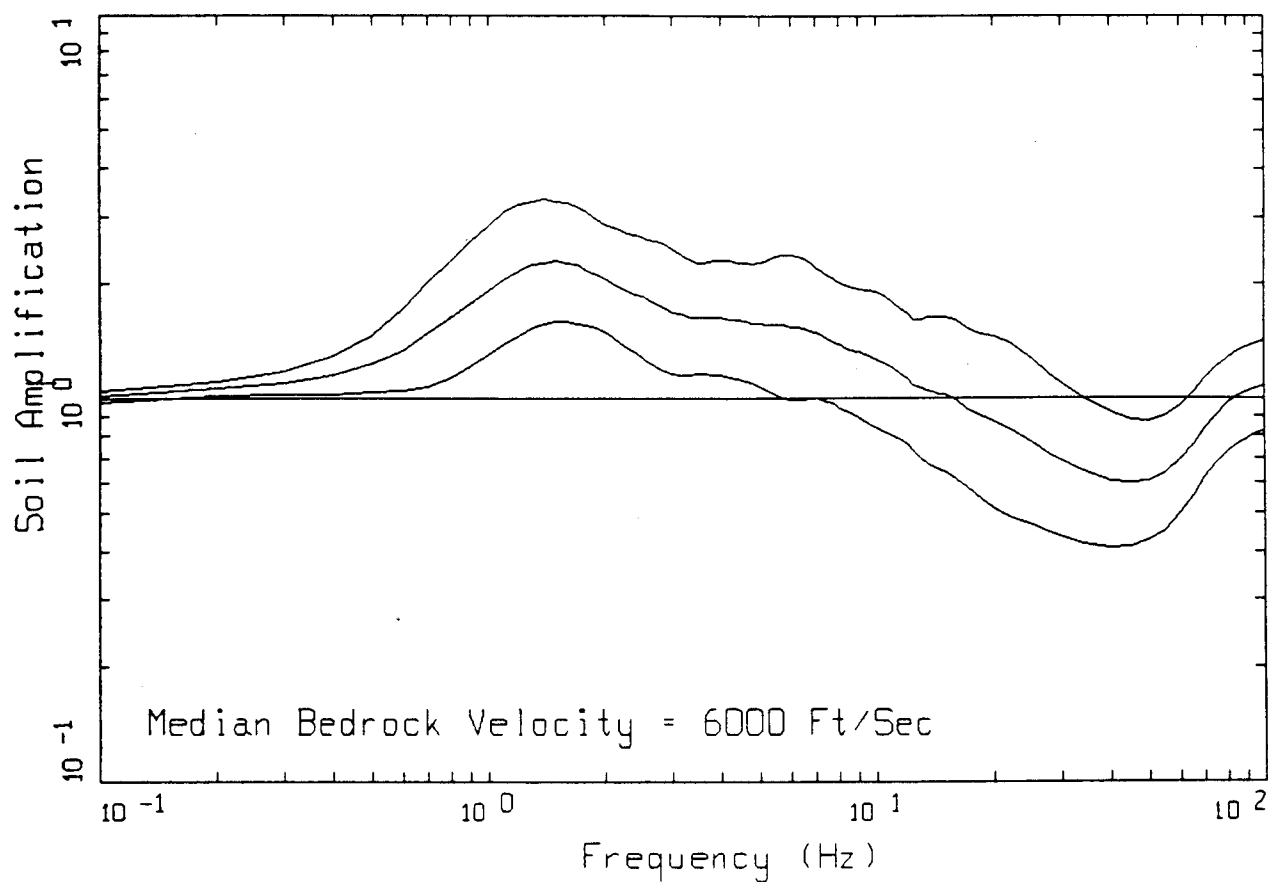


CATEGORY 4 (250 FT)

LEGEND	
—————	CONTROL MOTION 0.30 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
—————	CONTROL MOTION 0.30 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
—————	CONTROL MOTION 0.30 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.

Figure Set 6.C-4 (continued)

Category 4 (181–400 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

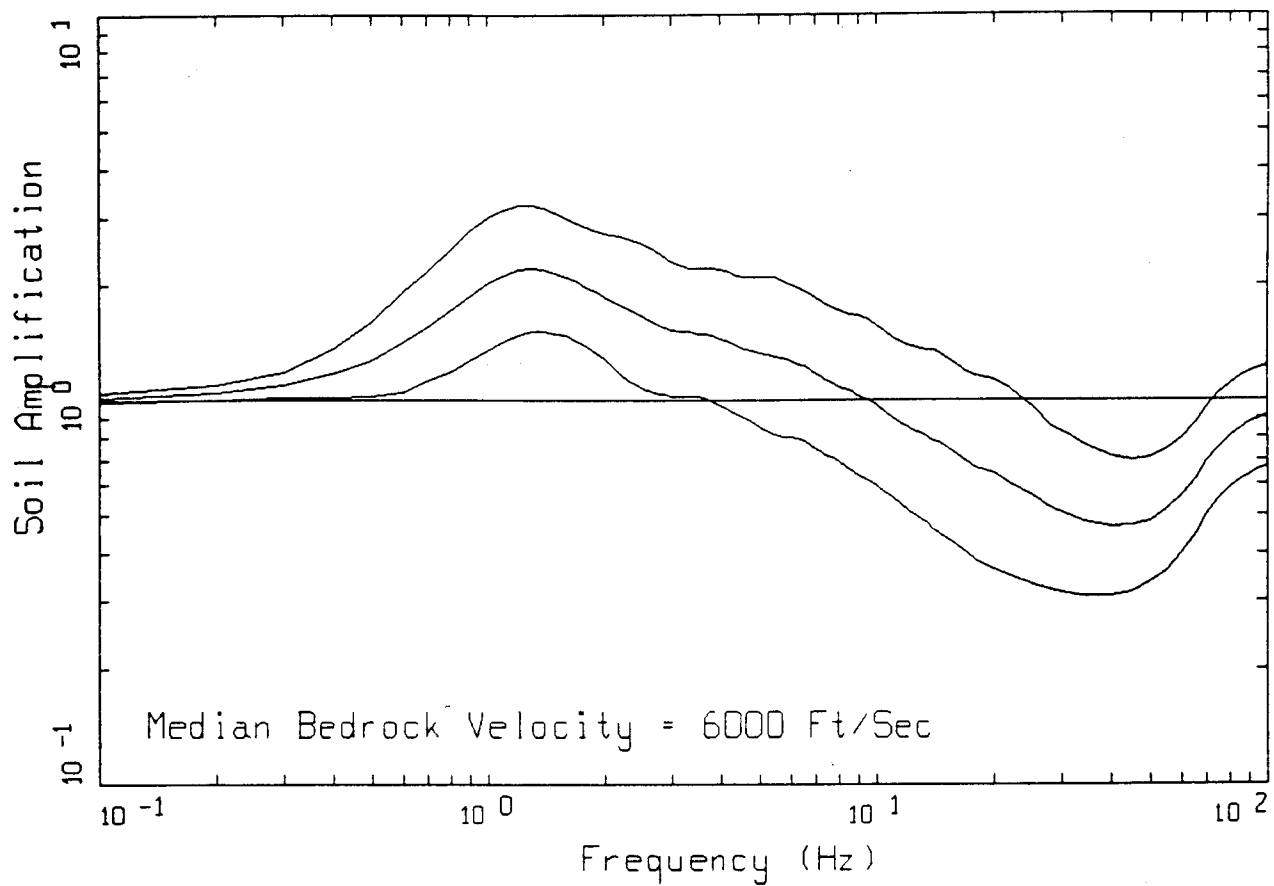


CATEGORY 4 (250 FT)

LEGEND	
—	CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
—	CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
—	CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.

Figure Set 6.C-4 (continued)

Category 4 (181–400 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

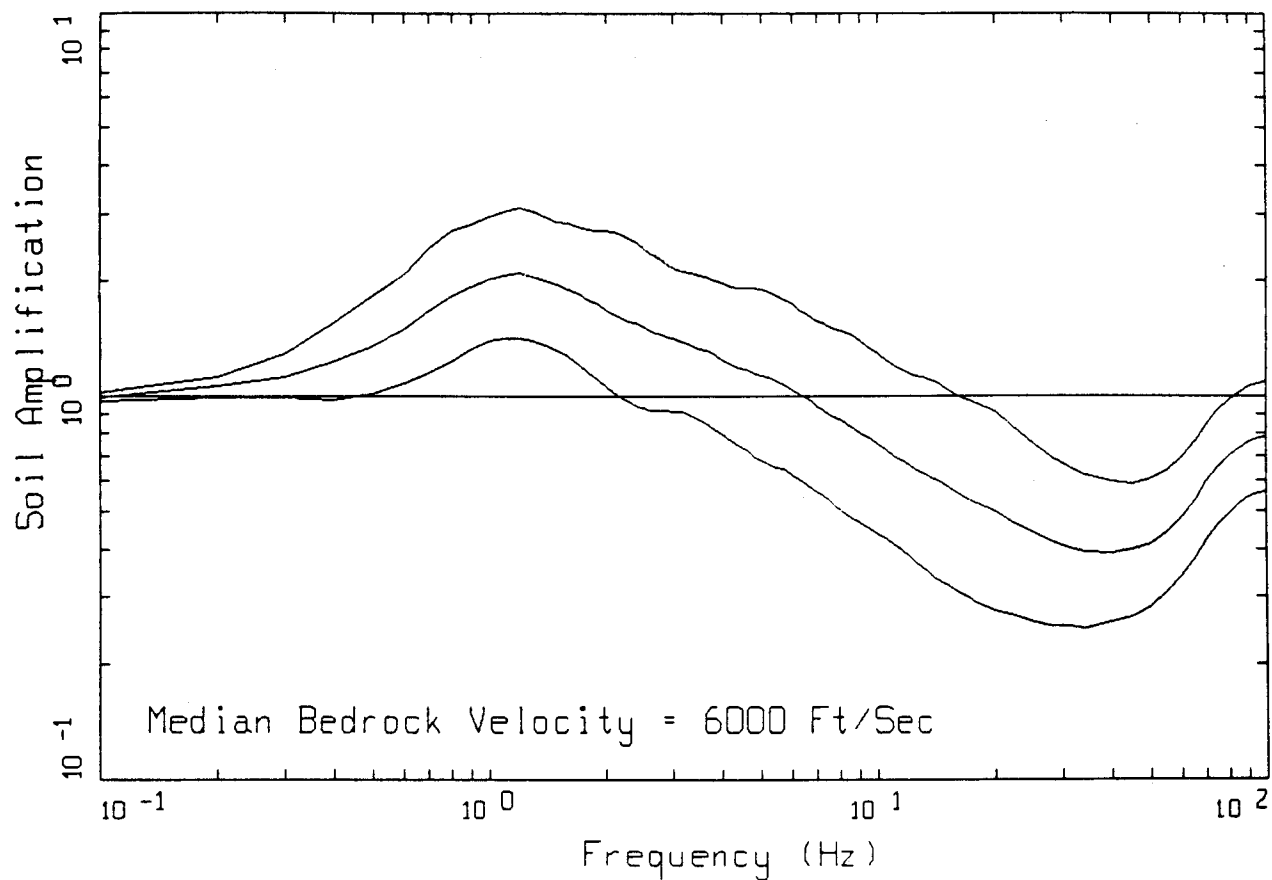


CATEGORY 4 (250 FT)

LEGEND	
—————	CONTROL MOTION 0.75 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
—————	CONTROL MOTION 0.75 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
—————	CONTROL MOTION 0.75 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.

Figure Set 6.C-4 (continued)

Category 4 (181–400 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

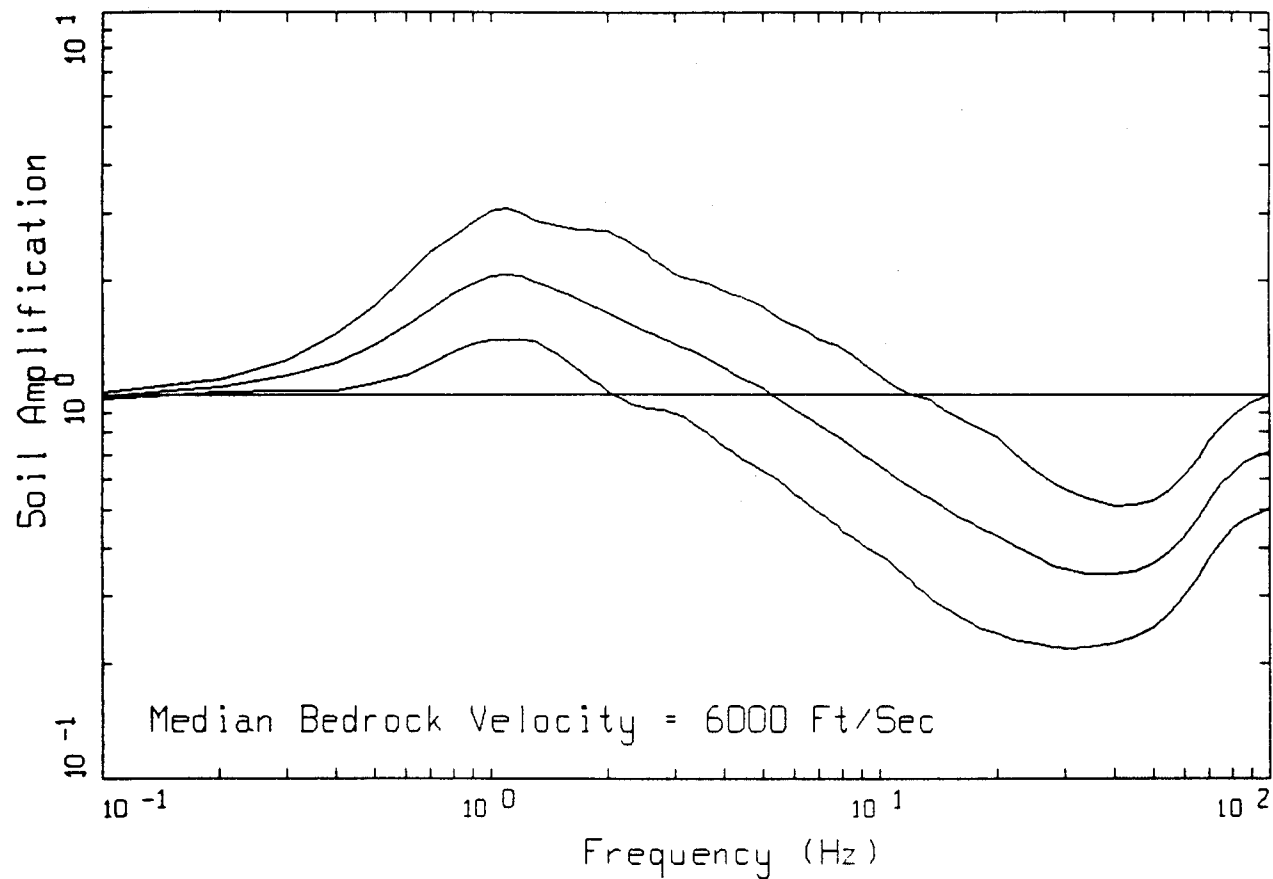


CATEGORY 4 (250 FT)

LEGEND	
————	CONTROL MOTION 1.00 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
————	CONTROL MOTION 1.00 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.
————	CONTROL MOTION 1.00 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 181 & 400 FT.

Figure Set 6.C-4 (continued)

Category 4 (181–400 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.



CATEGORY 4 (250 FT)

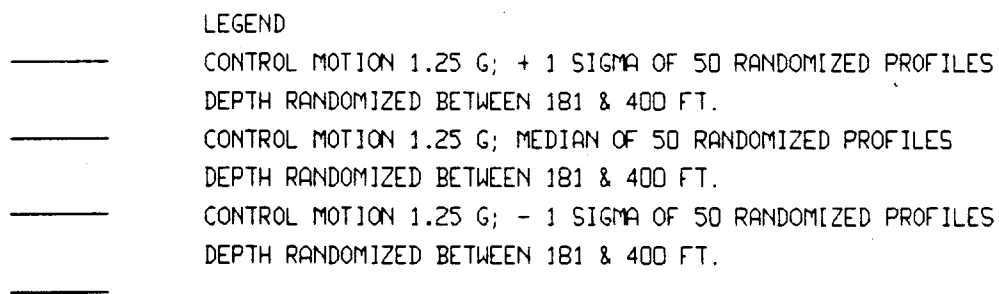
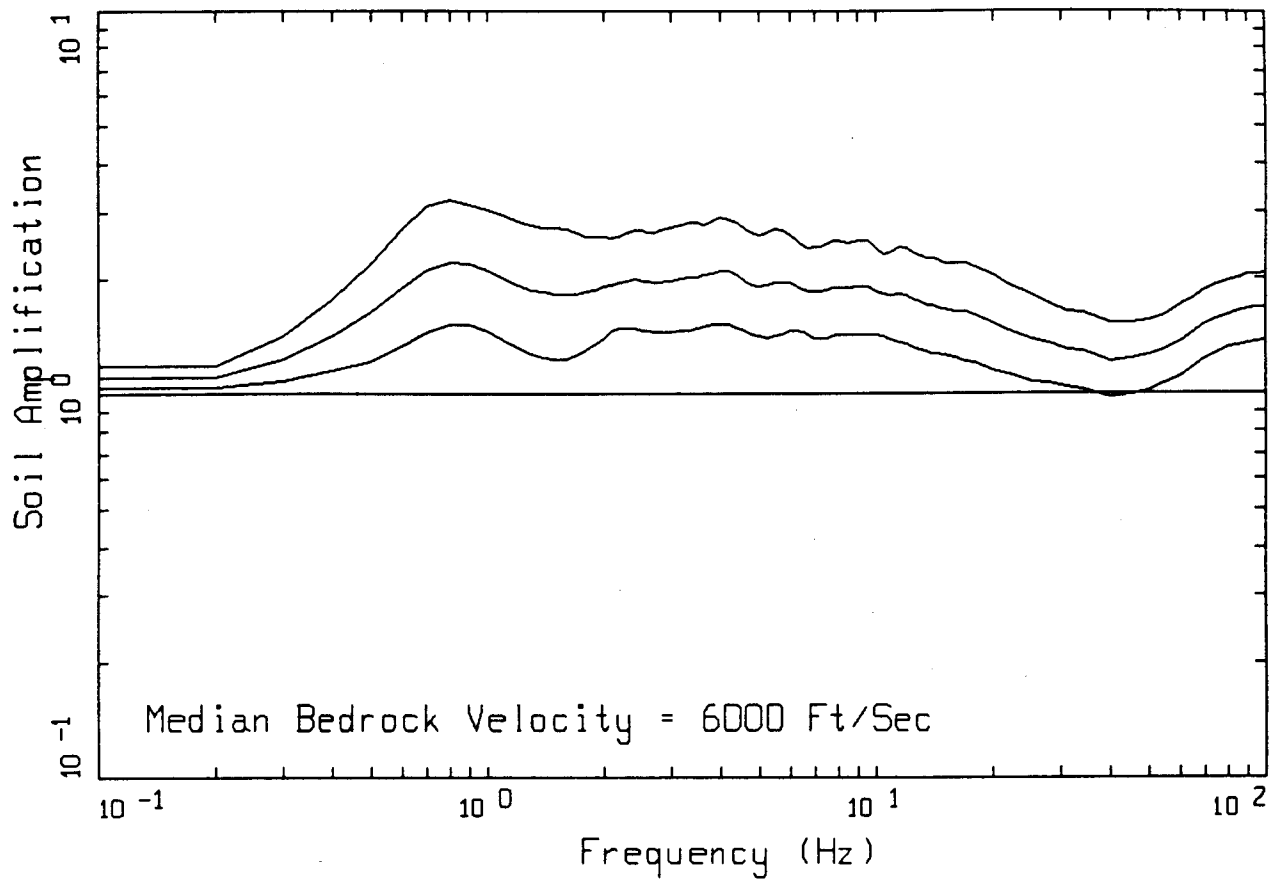


Figure Set 6.C-4 (continued)

Category 4 (181–400 ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

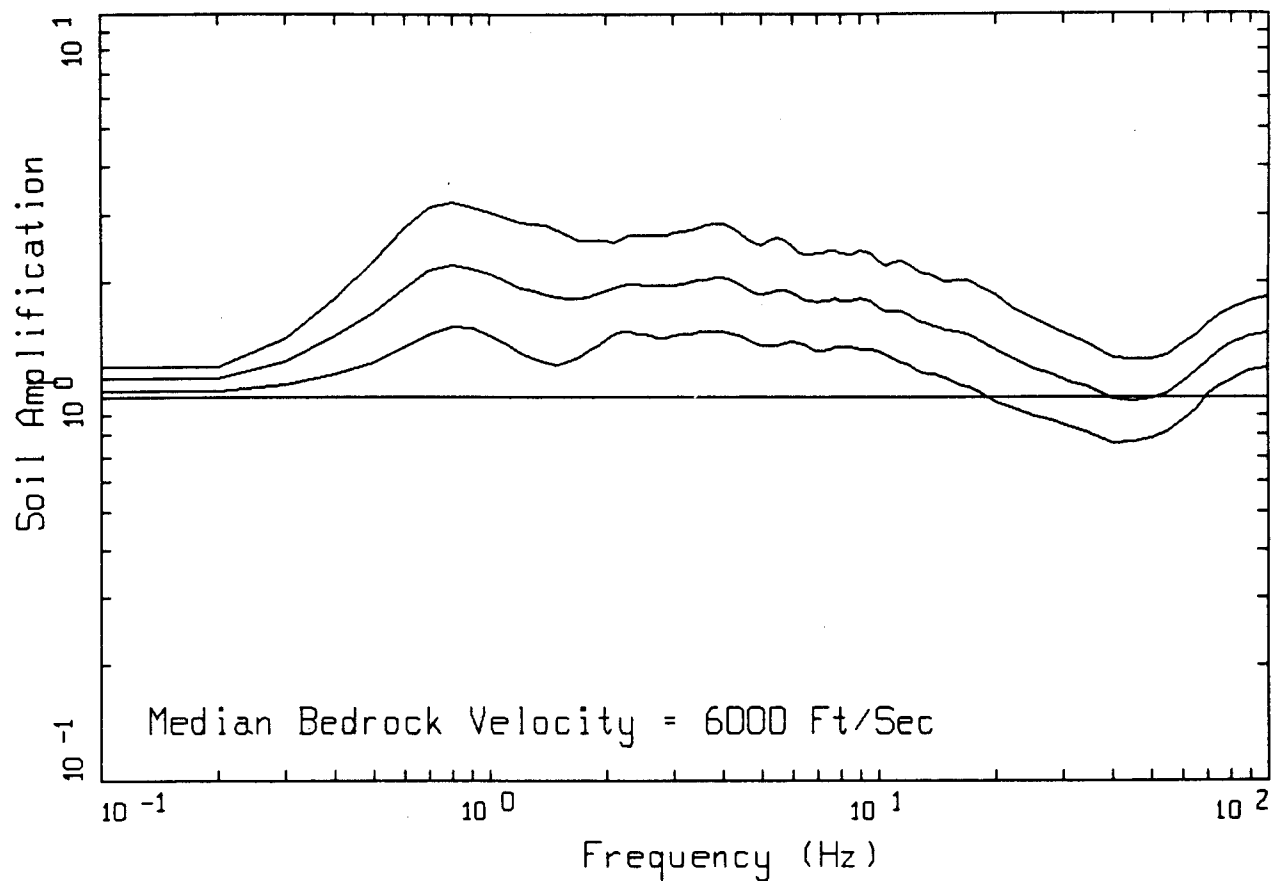


CATEGORY 5 (500 FT)

LEGEND	
————	CONTROL MOTION 0.05 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
————	CONTROL MOTION 0.05 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
————	CONTROL MOTION 0.05 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT

Figure Set 6.C-5

Category 5 (401+ ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

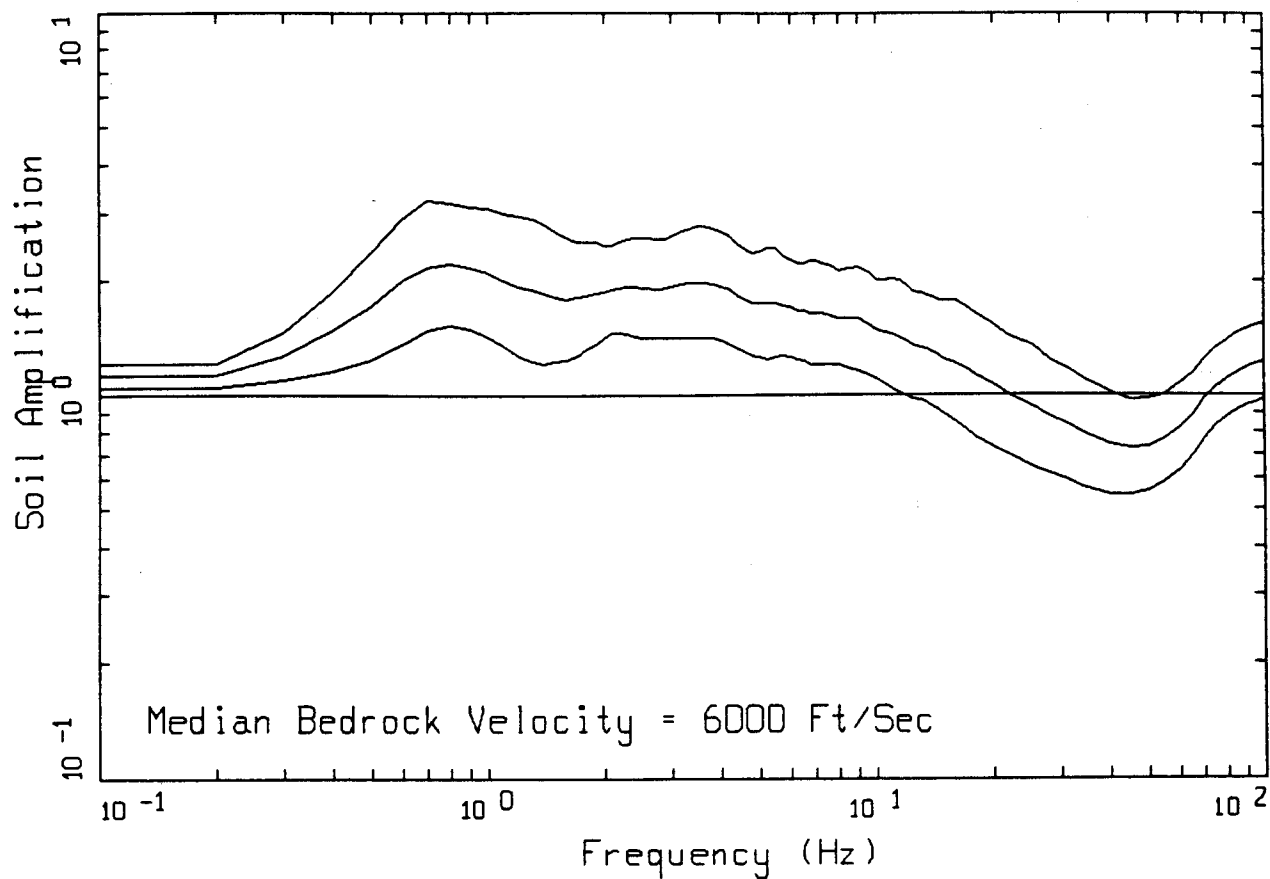


CATEGORY 5 (500 FT)

LEGEND	
—	CONTROL MOTION 0.10 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
—	CONTROL MOTION 0.10 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
—	CONTROL MOTION 0.10 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT

Figure Set 6.C-5 (continued)

Category 5 (401+ ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.



CATEGORY 5 (500 FT)

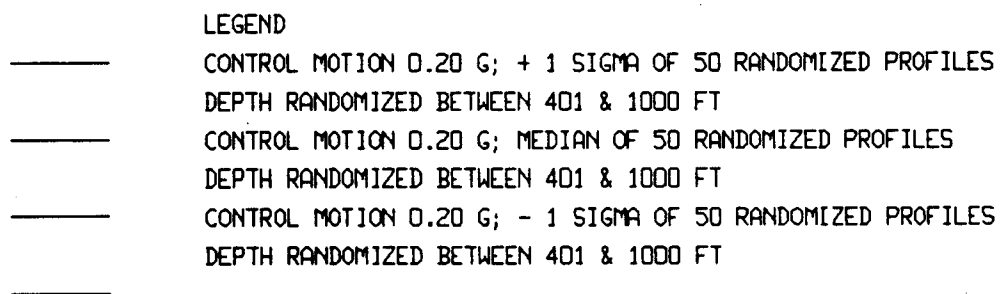
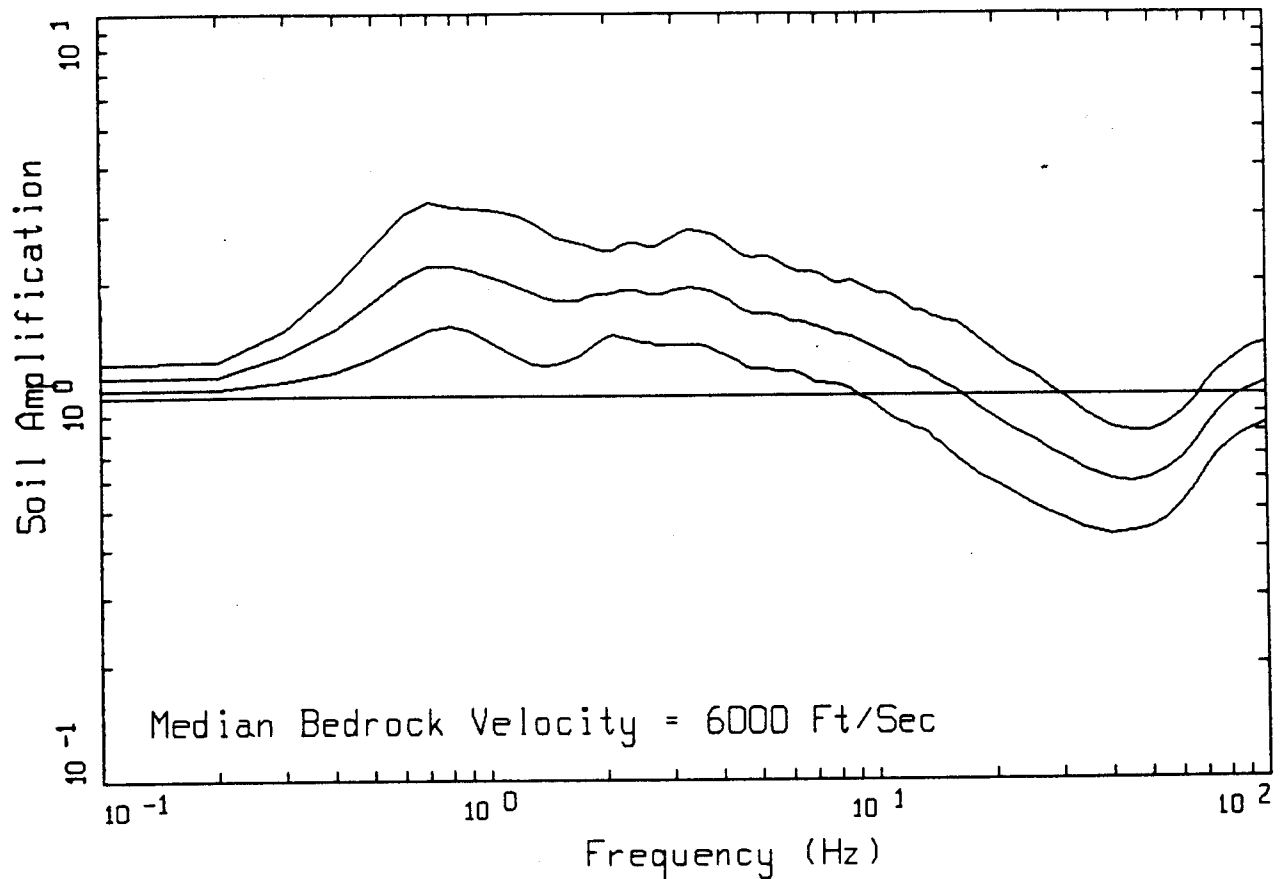


Figure Set 6.C-5 (continued)

Category 5 (401+ ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.



CATEGORY 5 (500 FT)

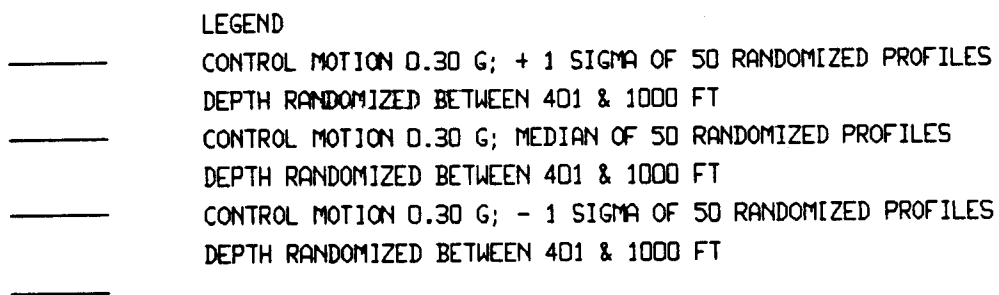
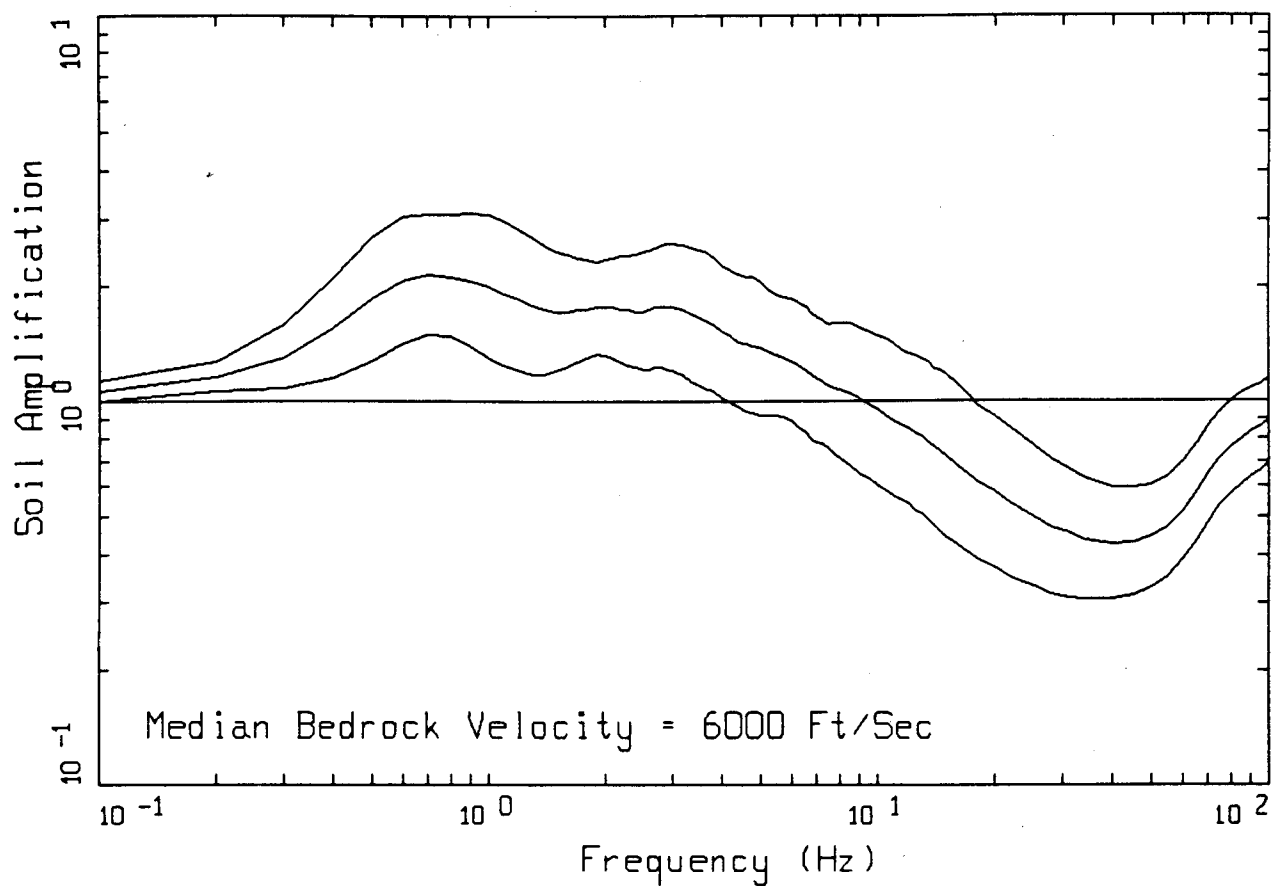


Figure Set 6.C-5 (continued)

Category 5 (401+ ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.



CATEGORY 5 (500 FT)

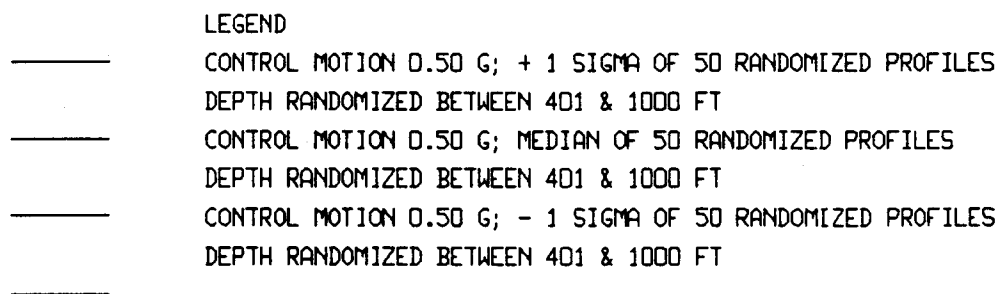
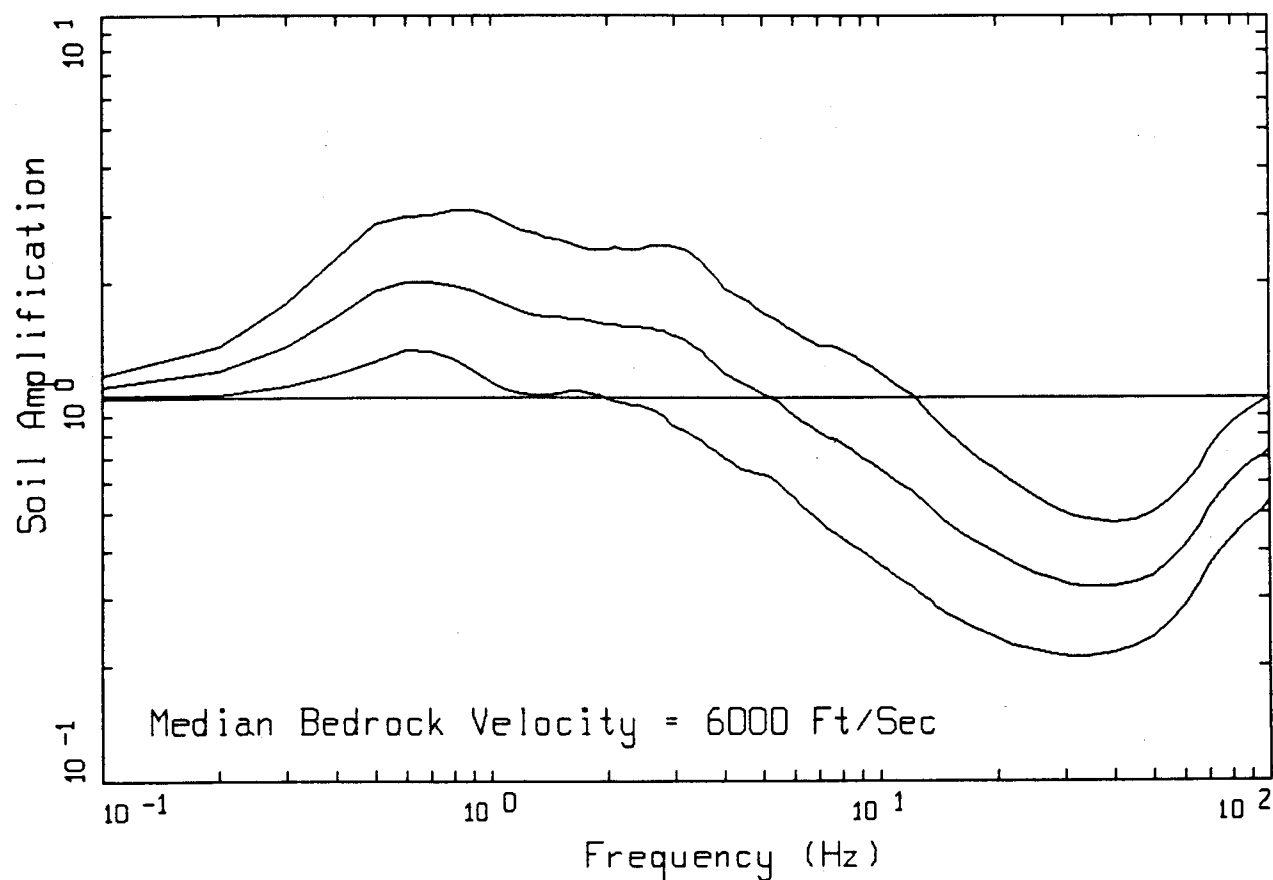


Figure Set 6.C-5 (continued)

Category 5 (401+ ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

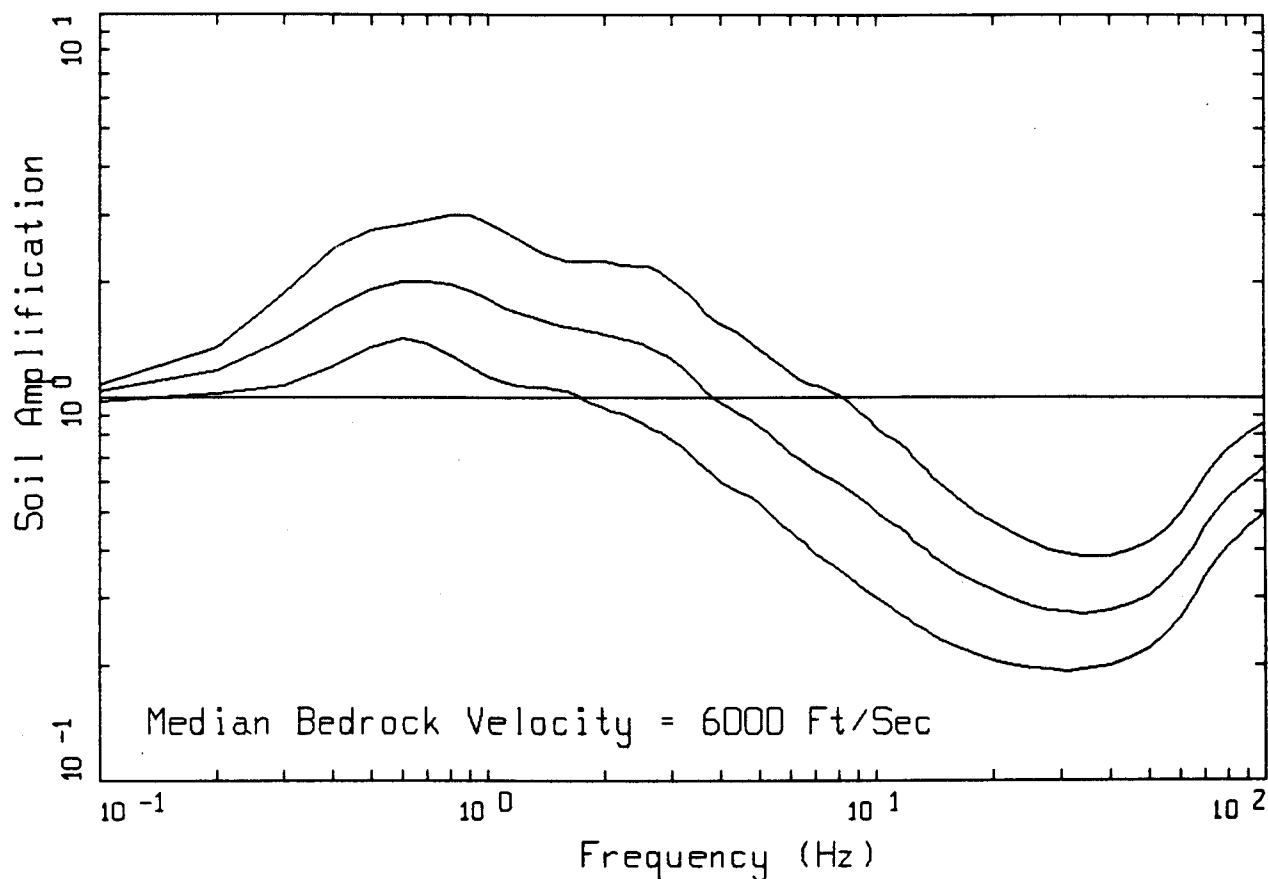


CATEGORY 5 (500 FT)

LEGEND	
—————	CONTROL MOTION 0.75 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
—————	CONTROL MOTION 0.75 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
—————	CONTROL MOTION 0.75 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT

Figure Set 6.C-5 (continued)

Category 5 (401+ ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

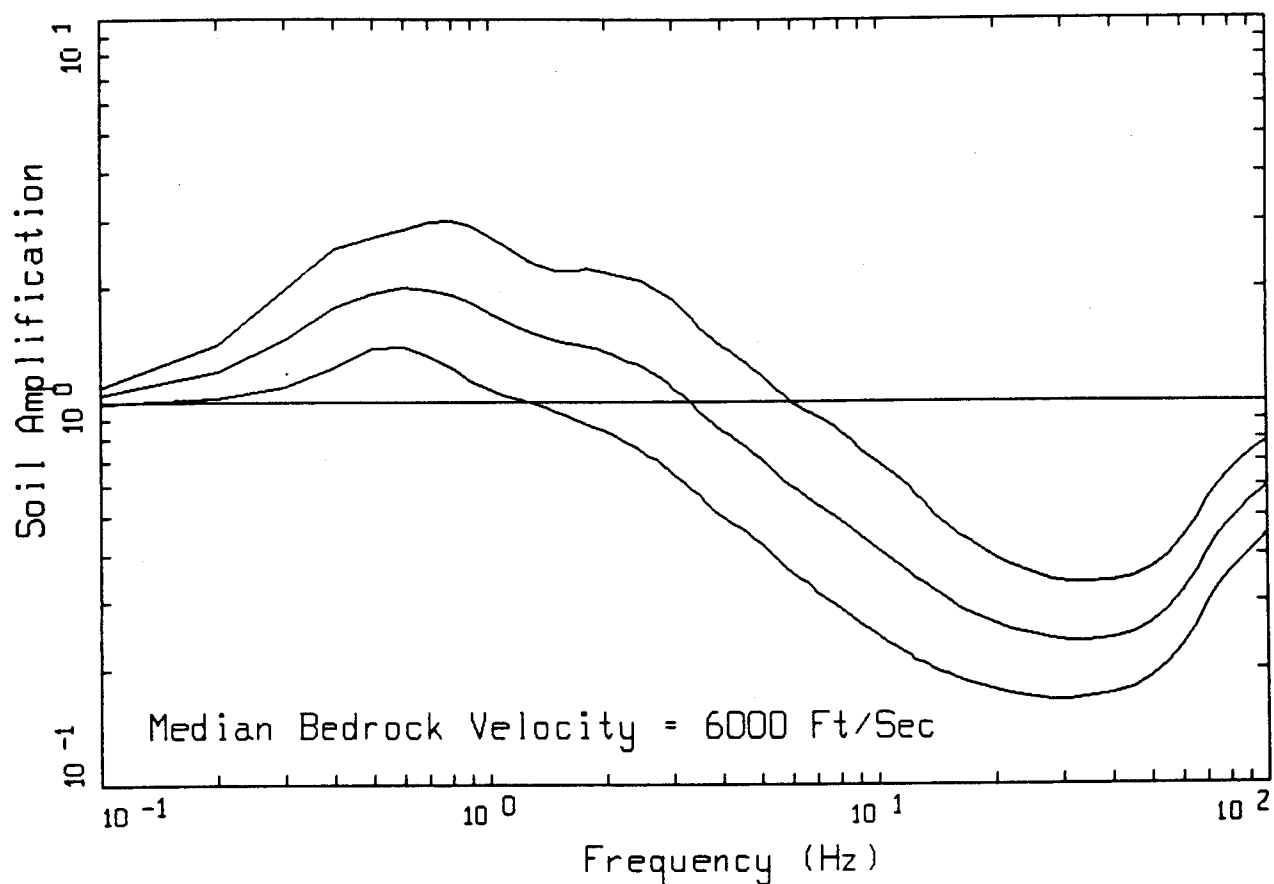


CATEGORY 5 (500 FT)

LEGEND	
—————	CONTROL MOTION 1.00 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
—————	CONTROL MOTION 1.00 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
—————	CONTROL MOTION 1.00 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT

Figure Set 6.C-5 (continued)

Category 5 (401+ ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.



CATEGORY 5 (500 FT)

LEGEND	
————	CONTROL MOTION 1.25 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
————	CONTROL MOTION 1.25 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
————	CONTROL MOTION 1.25 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
————	

Figure Set 6.C-5 (continued)

Category 5 (401+ ft) median and 1 sigma amplification factors for control motion peak acceleration levels of 0.05, 0.10, 0.20, 0.30, 0.50, 0.75, 1.00, and 1.25g. Median bedrock shear-wave velocity is 6000 ft/sec.

APPENDIX 6.D

ASSESSMENT OF COUPLING BETWEEN VERTICAL AND HORIZONTAL MOTIONS IN NONLINEAR SITE RESPONSE ANALYSIS

6.D.1 Introduction

This report presents the results of a numerical study performed to investigate the potential impact of nonlinear soil behavior on computed responses of soil sites when subjected to seismic inputs. The current procedures typically utilized to determine surface response is to perform iterated linear analyses of the soil column to horizontal inputs, in which the soils are modeled as linear materials, with shear moduli estimated from the results of cyclic laboratory experiments subjected to the same strains as computed in the column analysis. Nonlinear effects are typically accounted for by means of a linear hysteretic damping ratio selected to resemble the behavior of the laboratory experiments.

For vertical site responses, no general consensus has as yet evolved, although some recommend a similar convolution analysis based upon linear material modeling of the soil. These approaches make use of a vertical analysis performed uncoupled from the horizontal. Several important questions remain in this approach, namely,

- to what extent does material nonlinearity play a role in the vertical propagation problem,
- if material nonlinearity is important, how does coupling of the complete stress state impact the evaluation of the nonlinear behavior, and
- how should the vertical and horizontal response problems be coupled to account for the effects of this nonlinearity.

To try to address some of these issues, a calculational program was undertaken to study the response of two soil sites (Treasure Island and Gilroy) subjected to both horizontal and vertical input seismic motions. The motions used were those considered appropriate for the sites and recorded during the Loma Prieta event of October, 1989. The soil sites were represented by means of a finite element soil column extending upward from the top of bedrock to the ground surface, with the motions input to the base of the soil column (through equivalent linear dashpots to simulate elastic halfspace effects). Calculations of column response were then made for a number of different constitutive soil models (one isotropic elastic and two nonlinear) and for a number of different input and constraint conditions. A comparison of the surface responses from these different calculations are then used to address the issues mentioned above.

The calculations were made by means of a large finite element computer program (termed the SLAM Code) which has been in existence for a number of years, and which was developed to treat the wave propagation problem through nonlinear soil materials. A complete description of the code and the particular soil constitutive models selected for use in this study is presented in Section 6.D.2 of this report. The behavior of these soil models to static cyclic shear loadings was then undertaken to examine the behavior of these models under conditions considered appropriate to the seismic problem of interest. These results are presented in Section 6.D.3 of the report. The results obtained from the dynamic column studies are then presented in Section 6.D.4, with the conclusions and recommendations presented in Section 6.D.5.

6.D.2 Description of SLAM Code

This section of the report presents a summary of the computer program used to generate the results presented herein to investigate some effects of nonlinear material characteristics on the wave propagation characteristics of site soils subjected to seismic loadings. SLAM Code is a large two-dimensional finite element computer program originally developed for the U. S. Air Force (Refs. 7 through 10) to investigate ground shock propagation problems through soil and/or rock media induced by loadings developed from nuclear detonations. It was designed to treat problems in the mid to low pressure regimes where large finite strain effects are not of particular importance, but where the effects induced by nonlinear constitutive material laws are still significant. For those applications, it used as inputs surface overpressures at the ground surface as well as ground motions obtained from other codes which treated the high pressure regimes appropriate to the near-field region around the nuclear crater. For the seismic problem, the code has been used to investigate potential effects of material nonlinearities on the soil-structure interaction problem as well as other nonlinear phenomena associated with the seismic problem.

The acronym SLAM stands for Stress waves in Layered Arbitrary Media, with the term intending to amplify some of the principal features of the code. In general, the SLAM code treats two-dimensional motion through either axisymmetric or plane configurations. In its complete formulation, it has the capability to handle complex interior boundaries and soil layering configurations, embedded linear structures as well as quiet boundary conditions to enable treatment of large field problems with minimal boundary reflection effects. Of course, as indicated by its acronym, the code is able to handle a variety of nonlinear constitutive soil models. For this study, two different nonlinear material models were utilized, these being (1) a model satisfying the criteria of isotropic plasticity with a Coulomb-Mohr yield function and (2) a constitutive model developed to incorporate some well-known constitutive characteristics of soils such as nonlinear compaction effects as well as a variable yield function.

6.D.2.1 Problem Description

For the specific site soil column responses of interest herein, the code was suitably simplified to treat the finite element mesh shown schematically in Figure 6.D-1. The problem of interest considers upward propagation of SH waves through a number of soil layers, each capable of being represented by its own material model. Each soil layer is discretized into a number of finite elements, with the thickness of each element developed to enable transmission of frequencies up to a maximum defined in terms of the shear and compressional wave velocities by

$$f_{\max} = \frac{c}{4a} \quad 6.D-1$$

where c is the dilatational wave speed and a is the thickness of the element in the layer.

The equations of motion for all the nodes of the finite element mesh can be written symbolically as

$$M \ddot{x} = F^A - F^R \quad 6.D-2$$

where M is a diagonal mass matrix and x is the displacement vector considering both horizontal and vertical motions. For this simple plane problem, the horizontal and vertical motions of the nodes on both sides of the soil column are equal. The vector F^A represents the applied nodal loads while the vector F^R represents the resisting loads caused by the distortions of the elements adjacent to the nodes. The only applied loads in this problem are developed from the specified horizontal and vertical accelerograms input to the base of the soil column through the dashpot used to represent the elastic halfspace below the soil overburden. The resisting forces incorporate both the elastic and nonlinear components of the material constitutive models included in the analysis. The equations of motion are then integrated using a predictor/corrector integration scheme based on the Newmark beta method of integration. The time step utilized in this algorithm is selected from the maximum frequency transmission characteristics indicated in Equation 6.D-1.

The material constitutive behavior is handled in SLAM code through the following formulation. The stress-strain relations are written generically in the form

$$\{\sigma\} = [C] (\{\epsilon^T\} - \{\epsilon^N\}) \quad 6.D-3$$

where $\{\epsilon^T\}$ is the total strain vector $\{\epsilon_x, \epsilon_y, \epsilon_z, \gamma\}$, $\{\epsilon^N\}$ is the nonlinear strain vector, $\{\sigma\}$ is the stress vector defined by $\{\sigma_x, \sigma_y, \sigma_z, \tau\}$ and $[C]$ is the four by four elastic stress-strain matrix. In this notation, x and y represent the horizontal directions, z is the vertical direction and the shear is τ_{xz} . The total strains in the element are related to the nodal displacements through the relation

$$\{\epsilon^T\} = [B] \{X\} \quad 6.D-4$$

where $\{X\}$ is the vector of nodal displacements and B is a matrix defined by the element geometry. The element resisting forces are then computed from the element stresses computed from Equation 6.D-3 and distributed to the nodes to compute nodal accelerations at each time step of the integration. The description of the two material models considered in this study are presented in the following sections.

6.D.2.2 Coulomb-Mohr Material Model, Associated Flow Rule

One of the material models typically utilized to attempt to capture soil behavior is the plasticity model using the Coulomb-Mohr yield criterion. The constitutive relationship is based upon the Drucker-Prager (Ref. 11) formulation for isotropic plasticity in which a yield criterion is defined relating the stress invariants for the state of stress at a point. If the invariants of the stress state fall within the yield surface, the material is assumed to behave elastically. If the stress state is located on the yield surface, the material is assumed to have the capability to flow plastically, with the amount and direction of flow being related to the stress increment being applied to the soil element.

The yield surface for the Coulomb-Mohr model is defined by

$$f = \alpha J_1 + \sqrt{J_2'} = \kappa \quad 6.D-5$$

where J_1 is the first stress invariant given by

$$J_1 = (\sigma_x + \sigma_y + \sigma_z) \quad 6.D-6$$

and J_2' is the second invariant of the deviatoric stress components is given by

$$J_2' = [(\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2] / 6 + \tau^2 \quad 6.D-7$$

for the stress conditions of interest in this plane problem. In the notation used herein, x and y refer to the horizontal directions and z refers to the vertical direction. The shear stress τ is the shear in the x-z plane.

The parameters α and κ of Equation 6.D-5 are two material parameters which are related to the more familiar soil mechanics parameters c and ϕ obtained from a compressive triaxial test series and representing the cohesion and friction angle of the soil. For the condition of plane stress, these parameters are defined by

$$\alpha = \frac{2}{\sqrt{3}} \frac{\sin(\phi)}{(3 - \sin(\phi))} \quad \text{and} \quad \kappa = \frac{6c}{\sqrt{3}} \frac{\cos(\phi)}{(3 - \sin(\phi))} \quad 6.D-8$$

To simulate collapse under plane strain conditions, the parameters must be defined as

$$\alpha = \frac{\tan(\phi)}{\sqrt{(9+\tan^2(\phi))}} \quad \text{and} \quad \kappa = \frac{3c}{\sqrt{(9+\tan^2(\phi))}} \quad 6.D-9$$

If the current stress state lies within the yield surface defined by Equation 6.D-5, the material is assumed to behave elastically during the following strain increment. If the current stress state lies on the yield surface, the plastic strain rate vector is defined from the normality rule to the yield surface. Assuming small strain increments from the current state, the plastic strain increment can then be defined from

$$\{d\epsilon^P\} = \alpha \{I\} + \frac{1}{6\sqrt{J_2'}} [G] \{\sigma\} \quad 6.D-10$$

where $\{I\}$ is the unit vector defined by $\{1,1,1,0\}$ and $[G]$ is the symmetric matrix defined by

$$[G] = \begin{bmatrix} 2 & -1 & -1 & 0 \\ -1 & 2 & -1 & 0 \\ -1 & -1 & 2 & 0 \\ 0 & 0 & 0 & 6 \end{bmatrix} \quad 6.D-11$$

In principal stress space, the yield surface is a cone whose axis makes equal angles with the stress axes. If this axis is called the "A-line", the angle between the A-line and the normal to the yield surface is found from

$$\cos(\theta) = \left[\frac{\alpha^2}{\alpha^2 + 1/6} \right]^{0.5} \quad 6.D-12$$

The analysis proceeds by first assuming that no plastic flow occurs during the given increment in total strain. A fictitious stress state is then computed which can be defined as

$$\{\bar{\sigma}\}_i = [C] (\{\epsilon^T\}_i - \{\epsilon^P\}_{i-1}) \quad 6.D-13$$

If this new stress state lies below or on the yield surface, it is acceptable and is the true stress state. If it lies above the yield surface, it is inadmissible and the correct stress increment vector must be determined from the two criteria: (1) the new stress state must lie on the yield surface and (2) the plastic strain increment vector is proportional to the plastic strain rate vector defined by Equation 6.D-10. The new stress state is then found from

$$\{\sigma\}_i = \{\bar{\sigma}\}_i - \lambda [C] \{d\epsilon^P\}_{i-1} \quad 6.D-14$$

where $[C]$ is the elasticity matrix defined by:

$$[C] = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} (1-\nu) & \nu & \nu & 0 \\ \nu & (1-\nu) & \nu & 0 \\ \nu & \nu & (1-\nu) & 0 \\ 0 & 0 & 0 & \frac{1-2\nu}{2} \end{bmatrix}$$

6.D-15

for the plane strain problem of interest herein and λ is a factor of proportionality determined from the solution of a quadratic equation in terms of the current stress state and total strain increment. If the total strain increment is too large, a solution cannot always be found. In this case, iteration is used, dividing the total strain increment vector into smaller increments until a complete solution is obtained.

This analysis is based, of course, on the assumption that the initial stress state is not located at the apex of the cone of the yield surface. If it is, the plastic strain rate vector is undefined since the stress invariant J'_2 is zero. The analysis then proceeds as follows. It is first assumed that the stress state remains in the corner. Since the stresses would then remain constant during the increment in total strain, the entire strain increment occurs plastically. If the angle between this plastic strain increment and the A-line is less than the angle θ defined by Equation 6.D-12, this assumption is correct. If the angle is greater than θ , the stress state then moves out of the corner and the plastic strain rate vector is determined from the new stress state.

6.D.2.2.1 Coulomb-Mohr Material Model, Nonassociated Flow Rule. The use of the standard normality criterion to the yield surface to define the plastic strain increment leads to plastic volume expansion with nonlinear straining. This change in volume is typically greater than found from experiments on soils, particularly for the case of loose sands. On the other hand, the yield surface formulation of the Coulomb-Mohr model is a reasonable formulation which in a gross sense captures the strength characteristics of these soils. Thus, to maintain the yield formulation while reducing the amount of volume expansion associated with plastic flow, a nonassociated flow rule is used. From Equation 6.D-10, the total incremental volume change developed during plastic flow is found to be

$$dV^P = 3 \alpha \quad 6.D-16$$

For the nonassociated flow rule formulation, the plastic strain rate vector is defined as

$$\{d\epsilon^P\} = \beta \{I\} + \frac{1}{6\sqrt{J'_2}} [G] \{\sigma\} \quad 6.D-17$$

where β is a coefficient less than the yield parameter α . If β is 0, the flow rule is equivalent to the normality condition for a Mises material typically used to represent the yield function for frictionless materials such as ideal clays. The angle between the A-line and the plastic strain rate vector is also modified from Equation 6.D-12 to

$$\cos(\theta) = \left[\frac{\beta^2}{\beta^2 + 1/6} \right]^{0.5} \quad 6.D-18$$

The value of the angle β must be determined to approximately match experimental data (Refs. 4 and 5).

6.D.2.3 Compacting Soil Model

The compacting soil model used in these calculations is one of the variety of nonlinear models proposed and developed in the late 1960's and early 1970's at the U.S. Waterways Experiment Station (Refs. 2, 4 and 5) and incorporated into a number of ground shock computer codes (Refs. 1, 3 and 7). The objective of these codes was to treat the wave propagation problem through soil and rock materials associated with ground shocks developed from nuclear detonations. During the early development of these soil models, cyclic behavior of soil was not of particular interest. Rather, these calculations were primarily interested in studying the development of the high peak stresses developed in the ground at relatively close-in ranges to the center of such bursts. They therefore had to consider the highly nonlinear characteristic of the constitutive relations for the soil as well as the time details of the loading phenomena occurring within relatively short time periods of the arrival of the ground shock. The constitutive models were developed to match the stress-strain data developed from an extensive number of laboratory tests conducted under a variety of stress conditions (typically uniaxial and triaxial compression testing). Later developments (such as discussed in Ref. 6) extended these models to the longer time periods and lower stress levels associated with the cyclic behavior encountered during seismic events.

These soil models all incorporate the well-known property of irreversible compaction under hydrostatic loading as well as extend the concept of the yield function to include a variation of the yield strength with hydrostatic pressure. The approach expands the capability to match known experimental results, but requires the definition of many more parameters which must be determined from these experimental results. The yield surface description of the particular model used for this study is of the form

$$k_e = \kappa - \alpha J_1 \left(1 + \frac{J_1}{2c}\right), \text{ for } (J_1 + c) \geq 0$$

$$= \kappa + \frac{\alpha c}{2}, \quad \text{for } (J_1 + c) < 0$$
6.D-19

where (α, κ) are similar to the Coulomb-Mohr parameters described previously and c is an additional parameter which must be specified by input. The formulation is shown schematically in Figure 6.D-2 from which it may be noted that at low confining pressure the yield function approaches the Coulomb-Mohr model while at higher pressures (defined by the parameter c), the yield function approaches the Mises yield criterion. The parameter c then controls the shape of the yield function.

6.D.2.3.1 Shear Behavior. For stress states within the yield function, the shear behavior of the soil material for the model considered in these calculations is assumed to behave elastically; that is, the shear modulus is the initial elastic shear modulus, G_0 . For stress states on the yield surface, the potential for plastic flow exists, with the direction of the plastic strain rate vector defined by the normality condition assumed for the model. As with the Coulomb-Mohr model discussed previously, both associated and nonassociated flow rules were considered and are described in the following. The nonassociated plastic flow behavior is described first.

The first step in the analysis is to divide the total stress and strain vectors into their hydrostatic and deviatoric components or

$$\{\sigma\} = \{s\} + H\{I\}$$

$$\{\epsilon\} = \{e\} + E\{I\}$$
6.D-20

where $\{I\}$ is the unit vector $\{1,1,1,0\}$ as before. The stress-strain relation is then simply

$$dE = \frac{dH}{3K} \quad 6.D-21$$

and

$$\{de\} = \frac{1}{2G_0} \{ds\} + \lambda \{s\} \quad 6.D-22$$

where K is the bulk modulus and λ is the factor of proportionality to account for plastic flow during the strain increment. The invariant relationships are

$$J_1 = 3H \quad \text{and} \quad dJ_1 = 3 dH \quad 6.D-23$$

and

$$J_2' = -\frac{1}{2} \{s\}^T [F] \{s\} \quad \text{and} \quad dJ_2' = -\{s\}^T [F] \{ds\} \quad 6.D-24$$

where the superscript T refers to the vector transpose and $[F]$ is the symmetric matrix

$$[F] = \begin{bmatrix} 0 & 1 & 1 & 0 \\ 1 & 0 & 1 & 0 \\ 1 & 1 & 0 & 0 \\ 0 & 0 & 0 & -2 \end{bmatrix} \quad 6.D-25$$

The yield condition is then written as

$$J_2' = k_e^2 \quad \text{or} \quad dJ_2' = \beta dJ_1 \quad 6.D-26$$

where k_e is defined in Equation 6.D-19 and the term β is

$$\beta = 2 k_e \frac{dk_e}{dJ_1} \quad 6.D-27$$

Using the above relationships, the yield parameter λ may be found from

$$\lambda = \frac{1}{4 G_0 J_2'} \langle -\beta dJ_1 - \{s\}^T [F] \{ds\} \rangle \quad 6.D-28$$

where

$$\{\overline{ds}\} = 2 G_0 \{de\} \quad 6.D-29$$

and

$$dJ_1 = 9K dE \quad 6.D-30$$

In the computations, the value of λ is determined from Equation 6.D-28 using the values of stress and strain at the beginning of the time increment, together with the known values of the total strain increment. Again, iteration is used to refine the computation if the value of λ is too large.

The development of the model for the case of the associated flow rule is similar to the above development, except for the influence of plastic flow on the volume change and therefore on the hydrostatic stress. In this case, the stress-strain equations are modified to

$$dE = \frac{dH}{3K} - \lambda \frac{dk_e}{dJ_1} \quad 6.D-31$$

and

$$\{de\} = \frac{1}{2G_0} \{ds\} + \frac{\lambda}{2\sqrt{J'_2}} \{s\} \quad 6.D-32$$

The parameter λ is then computed from the relation

$$\lambda = \frac{-9K\beta dE - \{s\}^T [F] \{ds\}}{2G_0 + 9K\beta dk_e/dJ_1} \quad 6.D-33$$

6.D.2.3.2 Hydrostatic Behavior. As mentioned above, the series of compacting models incorporate the ability to account for nonlinear volumetric behavior and irreversible volume change under hydrostatic pressures. The particular model used for these calculations uses the following relationship between bulk modulus and hydrostatic pressures. On initial loading, that is, initial compression or decrease in J_1 , the bulk modulus is determined from either

$$K_L = (K_0 - \frac{\gamma c}{4} K_1) + \left[\frac{K_1}{\gamma c} (J_1 + \frac{\gamma c}{2})^2 \right] \quad \text{for } |J_1| \leq \gamma c \quad 6.D-34$$

or

$$K_L = K_0 - K_1(J_1 + \gamma c) + K_2(J_1 + \gamma c)^2 \quad \text{for } |J_1| > \gamma c \quad 6.D-35$$

where K_L is the loading bulk modulus and the parameters K_0 , K_1 , K_2 and γ are material parameters determined from experiment. For the unloading case defined by either a tensile increase in J_1 or a value of J_1 greater than a previously obtained value (preconsolidated state), the bulk modulus is defined by

$$K_U = K_0 - K_1 J_1 + K_2 (J_1)^2 \quad \text{for } (J_1 + J_1^L) \leq 0 \quad 6.D-36$$

or

$$K_U = K_4 \frac{(\kappa - \alpha J_1)^{0.5}}{(\kappa + \alpha J_1^L)^{0.5}} \quad \text{for } (J_1 + J_1^L) > 0 \quad 6.D-37$$

where

$$K_4 = K_0 + K_1 J_1^L + K_2 (J_1^L)^2 \quad 6.D-38$$

and J_1^L is another material parameter. A schematic representation of these bulk modulus relations are shown in Figure 6.D-2 and was developed to represent the behavior of typical granular materials. The parameter J_1^L is the value of the hydrostatic compressive pressure at which the material essentially behaves as a liquid. This is typically in the kilobar range, much beyond the levels of interest in these problems. No specification of the bulk modulus properties is made under tensile loadings since the material is assumed to have zero tensile capacity.

6.D.3 Cyclic Behavior of Soil Models

Prior to performing detailed time dependent wave propagation calculations for the soil columns of interest, the behavior of these constitutive relations under imposed cyclic static shear deformations was investigated. A number of calculations were performed for each soil model, straining samples at three depths within a fictitious soil column to investigate the effects of confinement on the cyclic behavior of these models. The elements were subjected to different levels of imposed maximum shear strain for up to ten complete cycles of loading. For each cycle, both average shear modulus and hysteretic damping ratios were computed. In each case, the soil was assumed to respond with no change in total volume to try to represent the behavior of a saturated soil.

6.D.3.1 Coulomb-Mohr Soil Model

The first calculation made was for a soil element located at a relatively shallow depth (about 37 feet below grade) within a soil column, corresponding to an effective average confining pressure of about 22 psi. The soil was assumed to have a shear wave velocity of 439.65 fps, a unit weight of 125 pcf and Poisson's ratio of 0.375, which properties correspond to the surface soil layer at the Treasure Island site. The soil is assumed to have a friction angle of 30 degrees and no cohesion. The corresponding parameters (α , κ) of the Coulomb-Mohr model described in Section 6.D.2.2 are (0.16, 0.0) respectfully. This soil element is then subjected to a number of cycles of shear strain at constant volume and the entire stress and strain history monitored to obtain effective degradation in shear modulus and effective hysteretic shear damping as would be done from the results of a laboratory experiment. The average shear modulus is computed as the average slope of the shear stress-shear strain curve from peak to peak and the hysteretic damping ratio as the hysteretic area/ $(2\pi\tau_{\max}\gamma_{\max})$.

The degradation properties of this model at the end of the first cycle of loading are shown in Figure 6.D-3 for three different cases of assumed normality condition of the plastic strain rate vector. The parameter B in this figure represents the ratio β/α of Section 6.D.2.2, or the degree to which the plastic strain rate vector satisfies the normality criterion. If B has the value of 0.0, plastic flow occurs with no corresponding plastic volume change (Mises flow condition) while if the parameter B has the value of 1.0, the associated flow rule requires plastic flow to be accompanied by volume expansion. The case of $B = 0.5$ is an intermediate flow condition.

As may be noted from Figure 6.D-3, the case of full nonassociated flow rule (no plastic volume change) leads to the largest decrease in the average shear modulus in one strain cycle as compared to those cases in which plastic volume change occurs during straining. Similarly, the case of full nonassociated flow rule leads to the highest hysteretic energy loss in one strain cycle. The degradation properties of the same model during additional cycles of shear straining is shown in Figure 6.D-4. As may be noted, the case of no plastic volume change leads to relatively constant cyclic properties. As soon as plastic volume change accompanies plastic flow, the amount of degradation decreases with each additional strain cycle, until finally no degradation occurs at later cycles in the test. This is explained by the fact that under conditions of constant total volume, any plastic volume expansion must be equilibrated by a corresponding elastic volume decrease, leading to an increase in compressive stresses in the element. For the Coulomb-Mohr model, this implies that the stress state moves further away from the yield surface with each cycle, until no additional plastic strain accompanies cyclic loading. Thus, after a number of cycles of strain, the sample effectively behaves elastically with no further energy loss per cycle.

Similar behavior is noted in Figures 6.D-5 and 6.D-6 for samples at deeper depths within the soil column with corresponding higher levels of initial confining pressures. At the depth of 107 feet, the effective average confining pressure is about 62 psi, while at a depth of 300 feet, the average confining pressure is about 173 psi. The results indicate that to achieve the same amounts of degradation at these deeper depths, the applied cyclic shear strains must increase. Figures 6.D-7 through 6.D-9 present comparisons of the effects of depth (or initial confining pressures) on the degradation during the first cycle of shear straining. As may be noted, the model develops a response qualitatively similar to that anticipated for actual soils; that is, the deeper the soil with corresponding greater initial confinement, the lower the amount of modulus degradation and damping ratio achieved with a given amount of straining.

6.D.3.2 Compacting Soil Model

Similar calculations were performed for the compacting model with the modified Drucker-Prager shear yield condition. As with all such models, the greater the number of parameters used to describe the constitutive behavior, the more difficulty in obtaining appropriate data to match particular responses. The data used for these calculations were obtained from studies of the behavior of alluvium in both uniaxial compression and triaxial test configurations. The primary difference between this model and that of the Coulomb-Mohr type is the considerable amount of hysteresis generated during uniaxial stress cycling. The different pressure-volume relations used to match measured bulk modulus data leads to this additional hysteresis.

The parameters G_0 and K_0 (initial shear and bulk modulus) were selected in all cases to match the initial shear and compressional wave velocities provided for the particular site being investigated. The remaining parameters of the soil model were selected to match the data available for the alluvium model described in Ref. 1. These values are summarized as follows:

K_1	= 8	K_2	= $4.76E-6$ 1/psi
α	= 0.16	κ	= 14.7 psi
c	= 36,750 psi	γ	= 0.15
J_1^L	= 176,400 psi		

The behavior of this soil model under the cyclic shear loading cycle at constant volume is similar to that described previously for the Coulomb-Mohr model. This would be expected for this loading case since the primary variant in this model from the classical plasticity model is its behavior under uniaxial stress states. The difference in behavior in cyclic shear stems from the modified form of the yield condition as compared to Coulomb-Mohr model. The results of the cyclic calculations are presented in Figures 6.D-10 through 6.D-14. Again, if plastic behavior with complete nonassociated plastic flow is incorporated into the model, relatively constant cyclic degradation behavior occurs with each load cycle. If some plastic volume expansion is allowed with plastic flow or yielding, the model essentially "shakes down" to the elastic state.

6.D.4 Stress Wave Calculations

The results of the SLAM code finite element soil column calculations for both the Treasure Island and Gilroy sites are presented herein. A number of different cases were investigated in an attempt to ascertain the sensitivity of the results to the various parameters controlling the vertical wave propagation characteristics. As mentioned previously, the calculations consider the vertical propagation of SH waves through sites with prescribed layering and elastic properties. The low strain properties were provided in terms of shear and compressional wave velocities. To ensure stability in the calculations, compressional wave velocities were reduced in a few cases so as to limit Poisson's ratio to values no greater than 0.48. Somewhat higher values could normally be accommodated, but for the purposes of this sensitivity study, this was not considered necessary.

For each of the two site soil columns analyzed, calculations were made for a number of different assumed conditions, including

- material models
 - elastic material model
 - Coulomb-Mohr material model
 - compacting material model
- input conditions
 - both horizontal and vertical motions
 - only horizontal motions
 - only vertical motions
- nodal restrictions
 - unrestricted vertical and horizontal motions
 - no nodal vertical motions allowed
 - no nodal horizontal motions allowed

The purpose of these various calculations was to try to assess sensitivity of the results to the details of the assumed problem conditions. For these calculations, a linear viscoelastic term was also included to attempt to simulate low strain energy dissipation, since, in general, the nonlinear models discussed previously will only provide energy dissipation capability at the higher strain levels. The viscous parameter was determined from the standard hysteretic damping relation

$$D = \frac{\eta \omega}{2G} \quad 6.D-39$$

where D is the hysteretic cyclic damping ratio, η is the equivalent viscous damping parameter, ω is the frequency of the cyclic motion and G is the equivalent shear modulus. For these problems, a 2% hysteretic damping ratio was used, which was assumed to be associated with a frequency of 20 Hz.

6.D.4.1 Treasure Island Results

The details of the layering assumed for the Treasure Island soil column are presented in Table 6.D-1. The overburden is specified as 300 feet deep and rests on an elastic halfspace with a shear wave velocity of 3222 fps. The soil column used in the calculations is defined by 11 soil layers with the elastic properties listed in the Table. The elements of the mesh were sized to be capable of transmitting at least 30 Hz. throughout the column. At the bottom of the mesh, the basement accelerograms were input through two pc dashpots to simulate the effect of the elastic basement materials. In the horizontal direction, the dashpot parameter is scaled to the shear wave velocity of the halfspace, while in the vertical direction it is scaled to the compressional wave velocity.

The input motions for these problems were taken as the Yerba Buena Island measured motions, channel 1 for the horizontal input and channel 2 for the vertical input. Plots of both time histories and corresponding 5% damped spectra for these input motions are shown in Figures 6.D-18 and 6.D-19. The corresponding measured motions in these two directions at the surface of the soil column are presented in Figures 6.D-20 and 6.D-21. The purpose of presenting these data is to indicate the character of the motions measured at the site and the order of magnitude of the induced amplification of the soil column. As may be noted from Figures 6.D-20 and 6.D-21, the measured horizontal motions indicate significant amplification over most of the frequency range while the vertical motions are deamplified. It should be noted, however, that no attempt has been made in this evaluation to address the issue of combination of horizontal motions.

Prior to performing the nonlinear time domain calculations, a standard iterated linear convolution calculation was performed to be used for comparison purposes. The computed surface responses obtained from these analyses are presented in Figures 6.D-22 and 6.D-23. For the horizontal calculation, the layering configuration and initial shear moduli conform to the data in Table 6.D-1. Soil degradation

properties for both modulus degradation and hysteretic damping ratios were the site specific Stokoe data provided for this study. For the vertical calculation, the soil column model matched the compressional wave velocities of Table 6.D-1, with no soil degradation properties included in the calculation.

The first time domain calculation performed was for the case of assumed elastic behavior of the soil column. Calculations were performed for the case of Poisson's ratio limited to 0.48 as mentioned above as well as 0.45. It was found that this small change in Poisson's ratio had negligible impact on either the horizontal or vertical motions computed at the ground surface. Four separate calculations were performed for this soil column, defined by the parameter KRUN, namely:

- KRUN=0—both horizontal and vertical motions are input to the column basement, and both motions are computed at each node in the soil column;
- KRUN=1—only horizontal motions are input to the soil column and both motions are computed at each node of the column;
- KRUN=2—only horizontal motions are input to the soil column and only horizontal motions are allowed at each node of the column;
- KRUN=3—only vertical motions are input to the soil column and only vertical motions are allowed at each node of the column.

The purpose of this set of problems was to attempt to assess the potential coupling of horizontal and vertical motions that is developed through the effect of Poisson's ratio. The horizontal motions developed at the ground surface for these cases are shown in Figure 6.D-24. It may be noted that these motions are essentially the same for each of the three cases run with computed horizontal input motions. They indicate little if any effect of vertical motion on the horizontal response. The peak accelerations computed are similar to those computed from the equivalent linear calculation shown in Figure 6.D-22 although the time domain calculations indicate that the high frequency content of the input motions (Figure 6.D-18) are transmitted directly up the column. This behavior, of course, is controlled by the amount of viscous damping incorporated into the calculation to represent low strain energy dissipation.

The corresponding vertical responses for these elastic calculations are shown in Figure 6.D-25. The motions in this case are similar to the motions calculated using the linear convolution model and again are not significantly affected by the horizontal motion assumptions through Poisson's effect. The corresponding response spectra for both the horizontal and vertical surface motions for these cases are shown in Figure 6.D-26. By comparison with the surface spectra computed from the frequency domain convolution calculations (Figures 6.D-22 and 6.D-23), the time domain calculations obtained from the linear material model are comparable, indicating that for this problem no significant modulus degradation develops in the frequency domain convolution calculation. Spectral ratios for these four cases are plotted in Figure 6.D-27, again emphasizing the lack of coupling between directions of motion through Poisson's effect.

The results for horizontal motions at the ground surface from the nonlinear calculations for the Treasure Island site using the Coulomb-Mohr material model are shown in Figure 6.D-28 for the same four column conditions defined above. The corresponding results for vertical motion are shown in Figure 6.D-29, with spectra for these cases plotted in Figure 6.D-30. The results are essentially the same as those found from the elastic problem (Figure 6.D-24 and 6.D-25) indicating that the combination of horizontal and vertical motions did not lead to movement of the stress state to the yield surface when horizontal shear stresses are developed from the horizontal input motions. Therefore, no nonlinear strains develop in this calculation.

If it is assumed that the initial in-situ compressive stresses do not change significantly during the seismic event, the shear strain which will initiate yield in the Coulomb-Mohr material model can be calculated as

$$\gamma_{\max} = \frac{1}{G} \left[(\kappa + 2 \alpha p_v)^2 - \frac{p_v^2}{12} \right]^{0.5} \quad 6.D-40$$

where (α, κ) are the Coulomb-Mohr shear parameters and p_v is the static vertical confining pressures at depth. It is assumed in Equation 6.D-40 that the horizontal pressures are equal to one-half the vertical. The maximum shear strains developed in the Treasure Island soil column are plotted in Figure 6.D-31, and are compared to the peak shear strains obtained from Equation 6.D-40. As may be noted, the column, to a first approximation, stays elastic throughout the motion.

The results of the Treasure Island computation with the compacting soil model are shown in Figures 6.D-32, 6.D-33 and 6.D-34. For this situation, the primary difference in the response from the elastic case is caused by the variability in the bulk modulus, since the previous information on strain levels indicates that shear strains do not cause the stress state to move to the yield surface, which would induce nonlinear shear deformations. A comparison of the peak horizontal accelerations (Figure 6.D-32) with the previous calculations (Figures 6.D-24 and 6.D-28) as well as the character of the computed surface motions indicates that the shear behavior of the soil column is essentially the same for the three analyses; that is, the initial shear modulus controls the horizontal calculations. Comparison of the vertical motions (Figure 6.D-33) with the results from the previous models (Figures 6.D-25 and 6.D-29) indicates that the vertical motions are more significantly influenced by the postulated nonlinear bulk modulus behavior. The peak vertical accelerations decrease from a value of 21.8 ips² to 15.4 ips² for the compacting model, although the general character of the motion appears similar.

The response spectra for the cases run with the compacting model are shown in Figure 6.D-34, which indicate the same general conclusions mentioned. At the frequency range between about 3 and 5 Hz, the peak of the vertical response spectrum is lower than that computed from the elastic soil column analysis. A comparison of the maximum shear strains developed in the column of compacting soil with those reached for the other soil models indicates that the peak shear strains increase by no more than 10% of their values computed elastically.

6.D.4.2 Gilroy Site Calculations

The Gilroy low strain elastic site properties used for these calculations are listed in Table 6.D-2. The computational profile consists of 17 soil layers over an elastic halfspace having a shear wave velocity of approximately 3900 fps, with the total depth of overburden of 560 feet to the top of "rock". By comparing the data in Table 6.D-1 and 6.D-2, the Gilroy site, in addition to being twice as deep, is composed of significantly stiffer soils than the Treasure Island site. The finite element column was again sized to allow transmission of frequencies of at least 30 Hz. The two basement dashpots had dashpot parameters defined by the halfspace properties of Table 6.D-2. Again, a small viscous damping parameter was included in the computation.

The horizontal acceleration history input to the base of the soil column is shown in Figure 6.D-35. By comparison with Figure 6.D-18, the input basement horizontal motion at Gilroy is significantly higher than at Treasure Island, with a peak acceleration of 0.44 g's (170.7 ips²) as compared to a peak of 0.067 g's (25.9 ips²) at TI. The basement accelerogram in the vertical direction at Gilroy is shown in Figure 6.D-36 and is an order of magnitude higher than that at TI (0.21 g's as compared to 0.027 g's). The motions measured at the surface at Gilroy are shown in Figures 6.D-37 and 6.D-38 for the horizontal and vertical directions respectfully. By comparison with Figures 6.D-20 and 6.D-21 for the TI site, the ratio of peak vertical to peak horizontal at Gilroy is significantly higher than recorded at TI. Again, these measured surface responses are shown merely to indicate the character of the responses but were not used further in this evaluation, since the column response analysis to three directional motions were not used in these calculations.

Prior to performing the time domain calculations, the iterated linear convolution analysis was performed to obtain surface responses in the normal fashion. In the horizontal direction, the elastic properties of Table 6.D-2 were used to define the low strain shear properties, and the Stokoe degradation properties were again used to obtain degraded moduli and hysteretic damping ratios. The surface motion and corresponding damped spectrum is shown in Figure 6.D-39. In the vertical direction a 2% hysteretic constant damping ratio was used with no iteration on degraded properties, with the computed surface motions and spectra presented in Figure 6.D-40.

As for the case of the TI calculations, the first set of time domain problems investigated was the elastic soil column, in which the low strain properties listed in Table 6.D-2 were used to define the column. The computed horizontal responses at the ground surface are shown in Figure 6.D-41 for the three assumed column conditions ($K_{RUN} = 0, 1$ and 2). The corresponding computed vertical surface motions are shown in Figure 6.D-42 for the three assumed column conditions ($K_{RUN} = 0, 1$ and 3). The computed response spectra for these cases are shown in Figure 6.D-43. As in the TI calculations, the differences in the results of these runs are negligible, indicating insignificant coupling between horizontal and vertical motions through Poisson's ratio effect. The peak horizontal acceleration (582 ips^2) in the time domain elastic calculation is significantly higher than that from the degraded frequency domain convolution calculation (294 ips^2) due to the higher damping as well as the reduced moduli included in frequency domain calculation. In the vertical direction, the peak accelerations and vertical spectra from the two calculations are similar.

Similar results for the cases of the columns modeled by the Coulomb-Mohr and compacting soil models are presented in Figures 6.D-44 through 6.D-48. Again, the results indicate some differences in peak responses and spectra but no fundamental differences in the character of the computed responses for any of the models. The peak shear strains computed in the soil column from the various analyses are shown in Figure 6.D-49. The results indicate that, by comparison to the shear strains computed from the approximate formula of Equation 6.D-40, nonlinear effects can be expected in the upper 300 feet of the soil column since the peak strains exceed the approximate yield strains in this region. Below this depth, it can be expected that the stress conditions will lie within the yield surfaces. However, it should be noted that the peak strains reached in the nonlinear models exceed those from the elastic case by relatively small amounts of the order of 10% to 15%. Thus the dynamic effects of the motion cause yielding but do not appear to allow sufficient time for these strains to grow unchecked for these cases with nonassociated flow rules.

6.D.4.3 Spectral Ratios

Based on the results of these computations, spectral ratios have been made for both horizontal and vertical motions for each site. The results for the Treasure Island site are shown in Figure 6.D-50 and the results for the Gilroy site in Figure 6.D-51. Spectral comparisons are made for the case of coupled horizontal and vertical motions ($K_{RUN}=0$) and compare the responses for the nonlinear models to the linear elastic. As may be noted, in the horizontal direction at both sites, the spectral ratios are essentially constant, with differences of no more than 10%. The horizontal motions are controlled by the shear modulus of the material and even at the higher acceleration levels at the Gilroy site does not appear to be significantly affected by nonlinear effects.

In the vertical direction, differences are somewhat larger although not dramatically so. At the Gilroy site, some discrepancies are noted at the higher frequencies, above about 15 Hz. At the Treasure Island site, the effect of compaction of the soil model appears to play a more important role, although it should be noted that the vertical motions at this site are an order of magnitude smaller than those at the Gilroy site.

6.D.5 Summary and Conclusions

This report has presented the results of nonlinear finite element calculations for the transmission of upward propagating SH waves, in an attempt to characterize the significance of the material nonlinearities on the characteristics of the resulting surface motions and the coupling of the horizontal

and vertical motions on this response. Two different nonlinear soil models were used to represent the characteristics of real soil constitutive behavior under bidirectional loadings. Comparisons of the computed responses were then made with those from the linear elastic representation of the soil. Calculations were made for soil columns representing both the Treasure Island and Gilroy sites, sites which have different site characteristics and were subjected to seismic inputs of different magnitudes.

The results of the calculations indicate the following.

- The horizontal surface response of the soil columns are essentially uncoupled from the vertical response for all three material models (linear, standard Coulomb-Mohr and compacting) at both sites.
- The horizontal shear strains developed for the nonlinear cases for either soil column are only slightly larger (within 10% to 15%) than those developed from the linear calculations.
- The vertical motions do not appear to be significantly coupled to the horizontal motions and only appear to be dependent on the bulk properties of the soil motions (load-unload characteristics of the materials). At the stiffer site subjected to the higher input motions, the difference between the linear and nonlinear problems did not appear to be significant.

On the basis of these results, it appears reasonable to conclude that the site response analyses, within an acceptable degree of tolerance, can be performed by uncoupling the horizontal and vertical analyses, and that the vertical motions can be estimated from soil properties determined from the uniaxial confined compression test.

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Table 6.D-1
Treasure Island Soil Column

Soil Layer No.	Shear Wave Velocity (fps)	Poisson's Ratio	Layer Thickness (Ft)	Bottom Depth (Ft)	Element Thickness (Ft)
1	439.65	0.375	8.01	8.01	2.67
2	439.65	0.480	18.24	26.25	3.65
3	557.77	0.480	17.39	43.64	4.35
4	492.15	0.480	9.84	53.48	3.28
5	492.15	0.480	18.01	71.49	3.60
6	492.15	0.480	10.29	81.78	3.43
7	577.46	0.480	12.72	94.50	4.24
8	1040.08	0.472	40.69	135.19	8.14
9	876.03	0.480	110.90	246.09	6.93
10	1266.47	0.475	42.65	288.74	8.53
11	2234.36	0.413	13.12	301.86	13.12
HALFSPACE	3221.94	0.442			

Table 6.D-2
Gilroy Soil Column

Soil Layer No.	Shear Wave Velocity (fps)	Poisson's Ratio	Layer Thickness (Ft)	Bottom Depth (Ft)	Element Thickness (Ft)
1	524.96	0.410	21.00	21.00	4.20
2	689.01	0.400	14.00	35.00	4.67
3	853.06	0.390	10.33	45.33	5.16
4	1000.02	0.390	11.39	56.72	5.70
5	1500.01	0.360	15.00	71.72	7.50
6	853.03	0.480	34.24	105.96	6.86
7	951.49	0.480	20.00	125.96	6.67
8	1214.74	0.450	20.00	145.96	10.00
9	1590.70	0.470	23.04	169.00	11.52
10	1975.02	0.450	15.03	184.03	15.03
11	2230.00	0.440	35.93	219.96	17.97
12	2230.00	0.420	12.69	232.65	12.69
13	1804.55	0.450	27.47	260.12	13.73
14	1804.55	0.430	47.74	307.86	11.94
15	1804.55	0.450	30.00	337.86	15.00
16	2165.46	0.420	27.61	365.47	13.81
17	2165.46	0.440	194.90	560.37	17.72
HALFSPACE	3899.99	0.420			

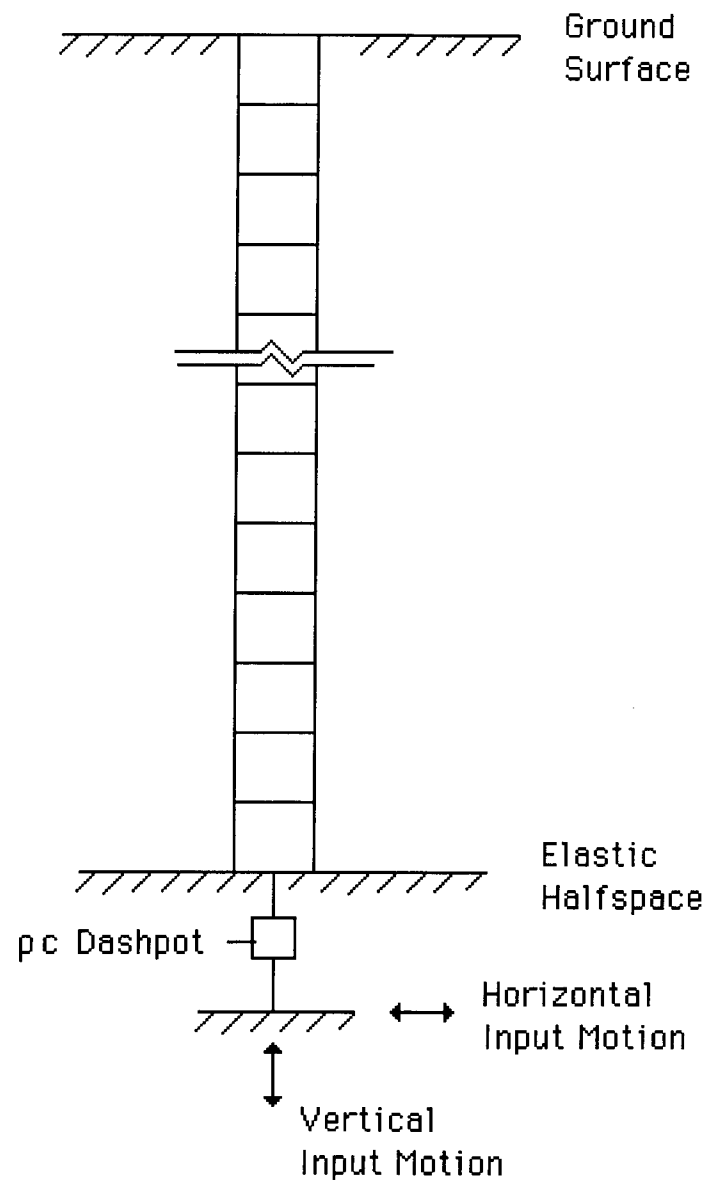
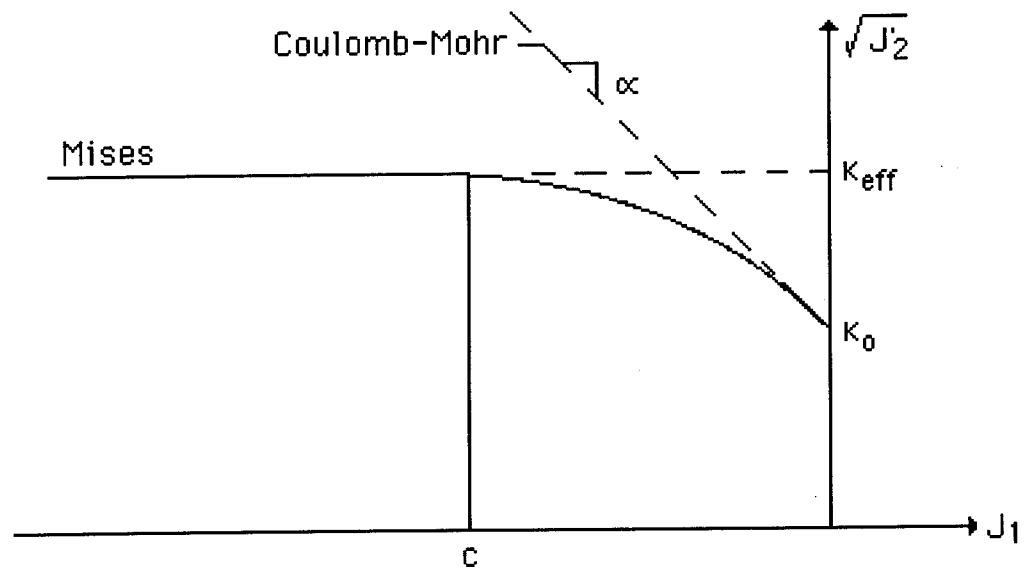
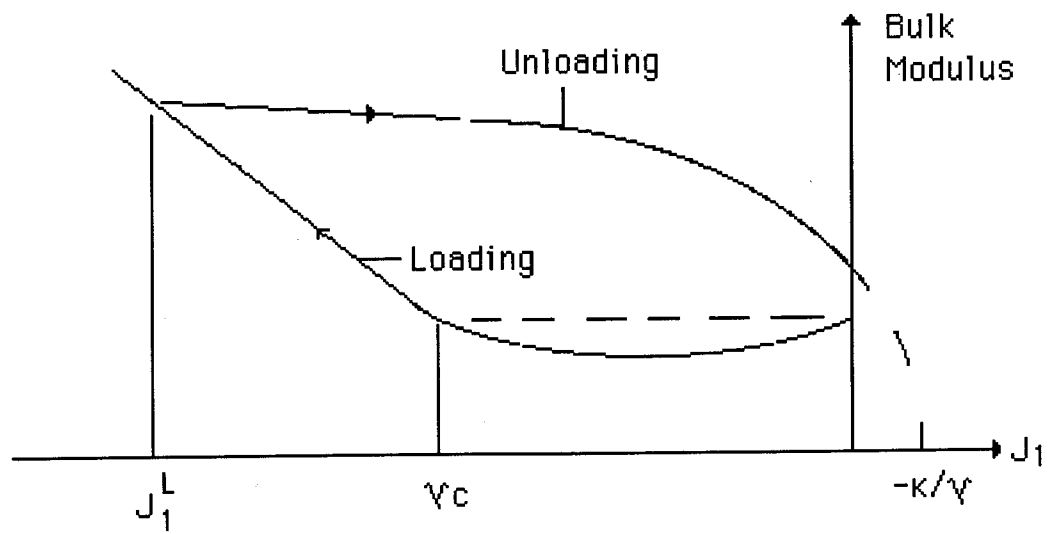


Figure 6.D-1
Finite element mesh representation of soil column



Variable Yield Function for Compacting Model



Bulk Modulus Function for Compacting Model

Figure 6.D-2
Characteristics of compacting soil model

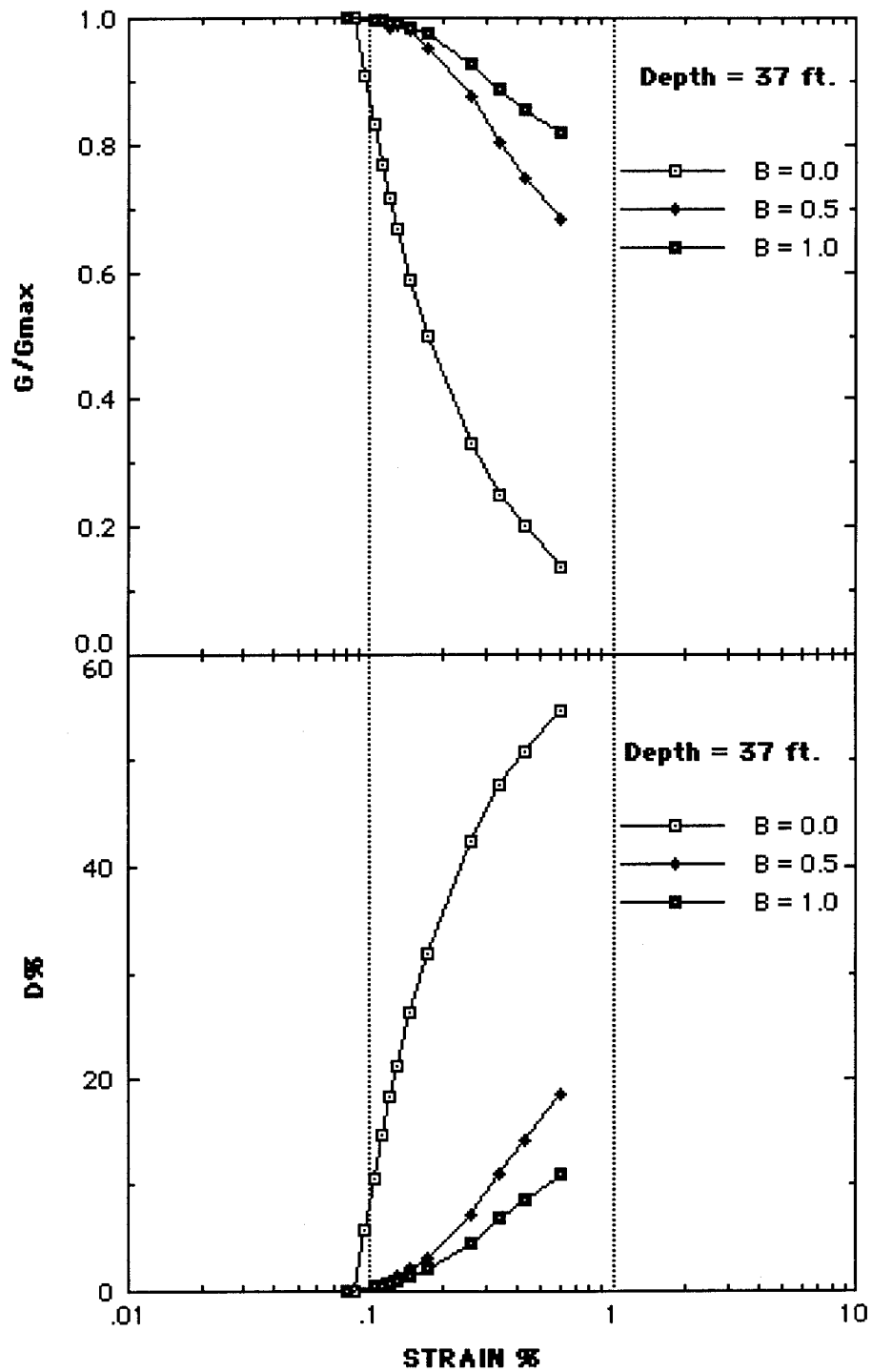


Figure 6.D-3
Cyclic behavior of Coulomb-Mohr material at shallow depth

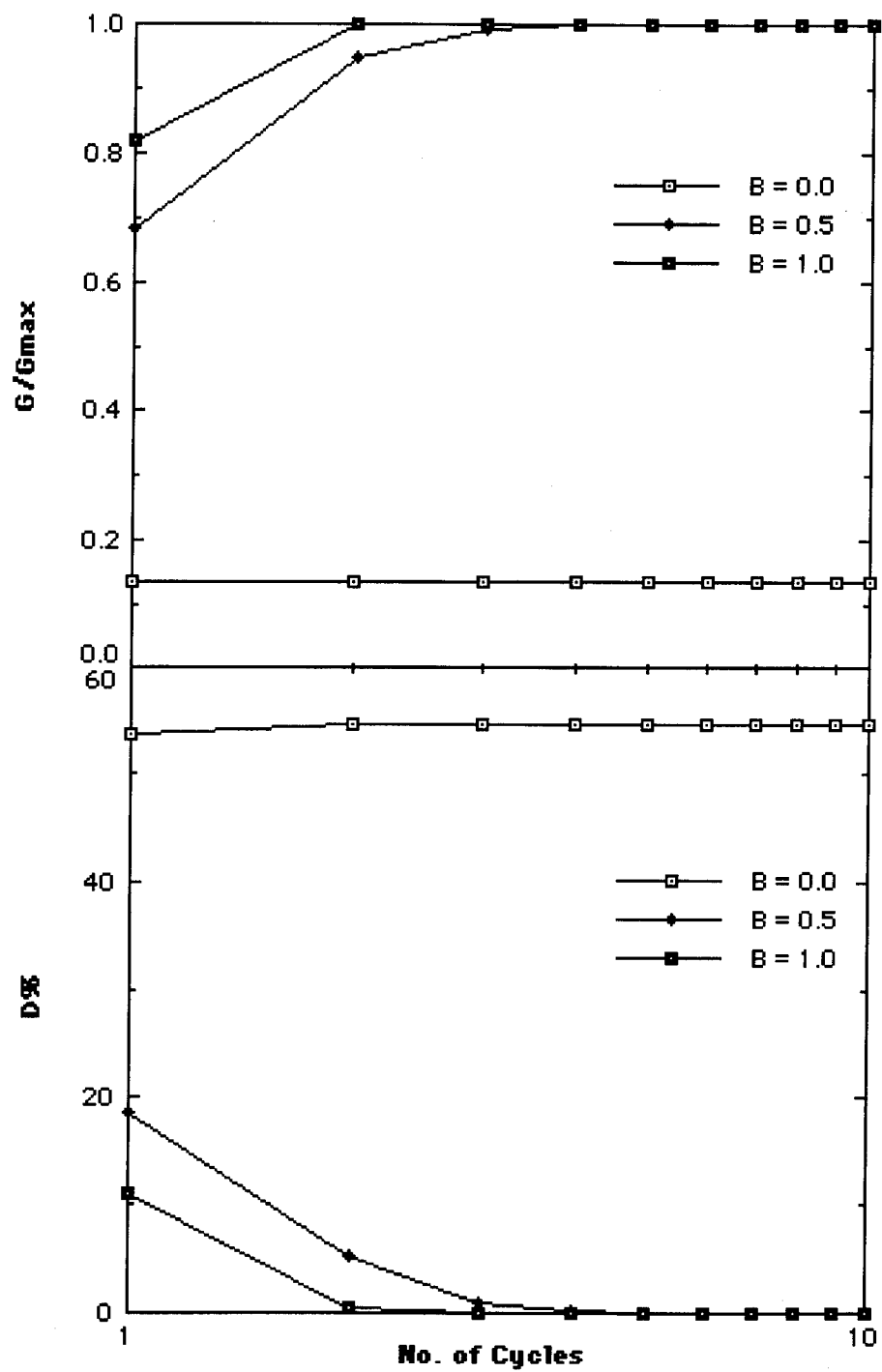


Figure 6.D-4

Change in degradation properties with cycle of strain for Coulomb-Mohr material at shallow depth (37'), max. cyclic shear strain=0.6%

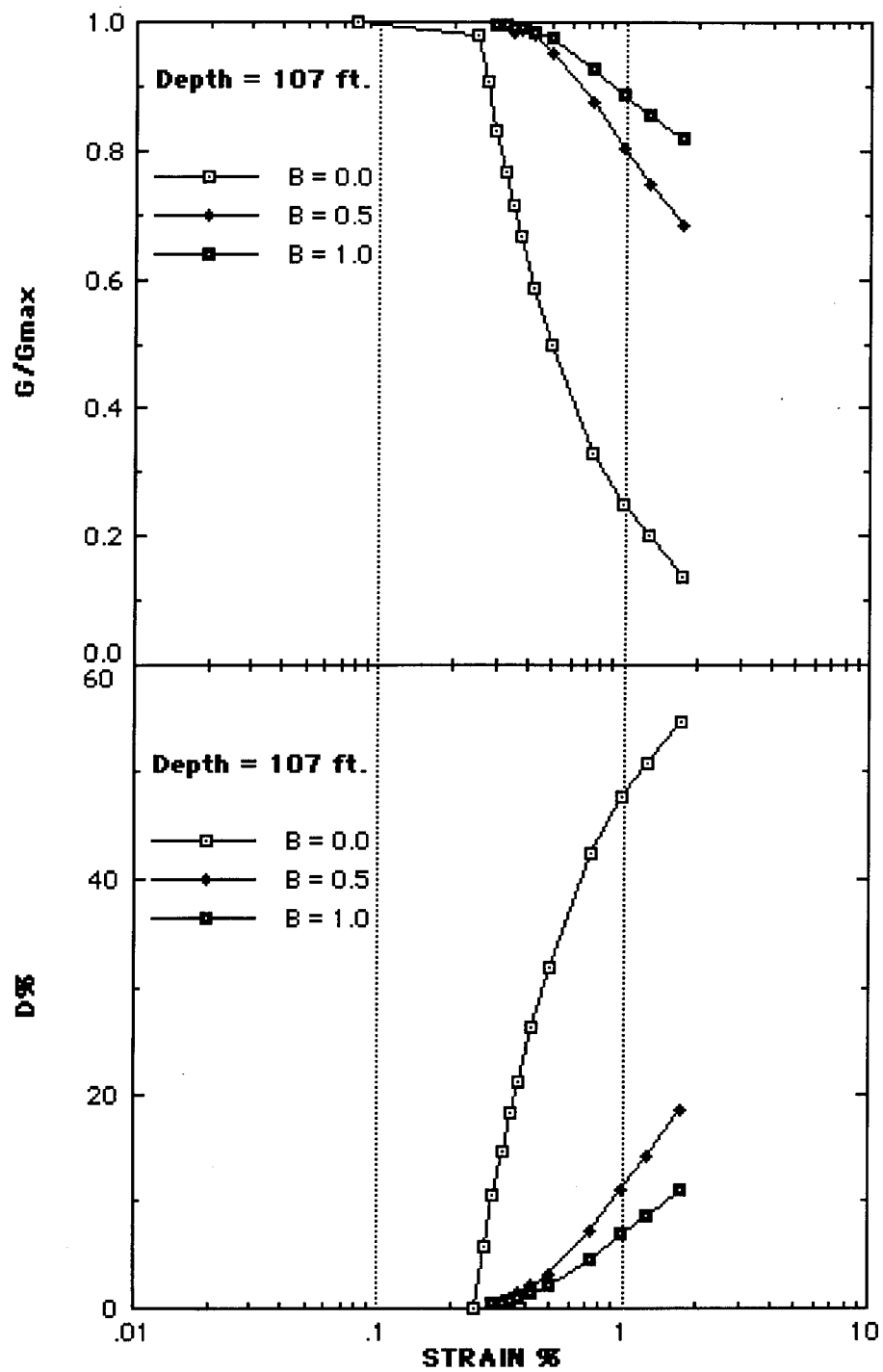


Figure 6.D-5
Cyclic behavior of Coulomb-Mohr material at intermediate depth

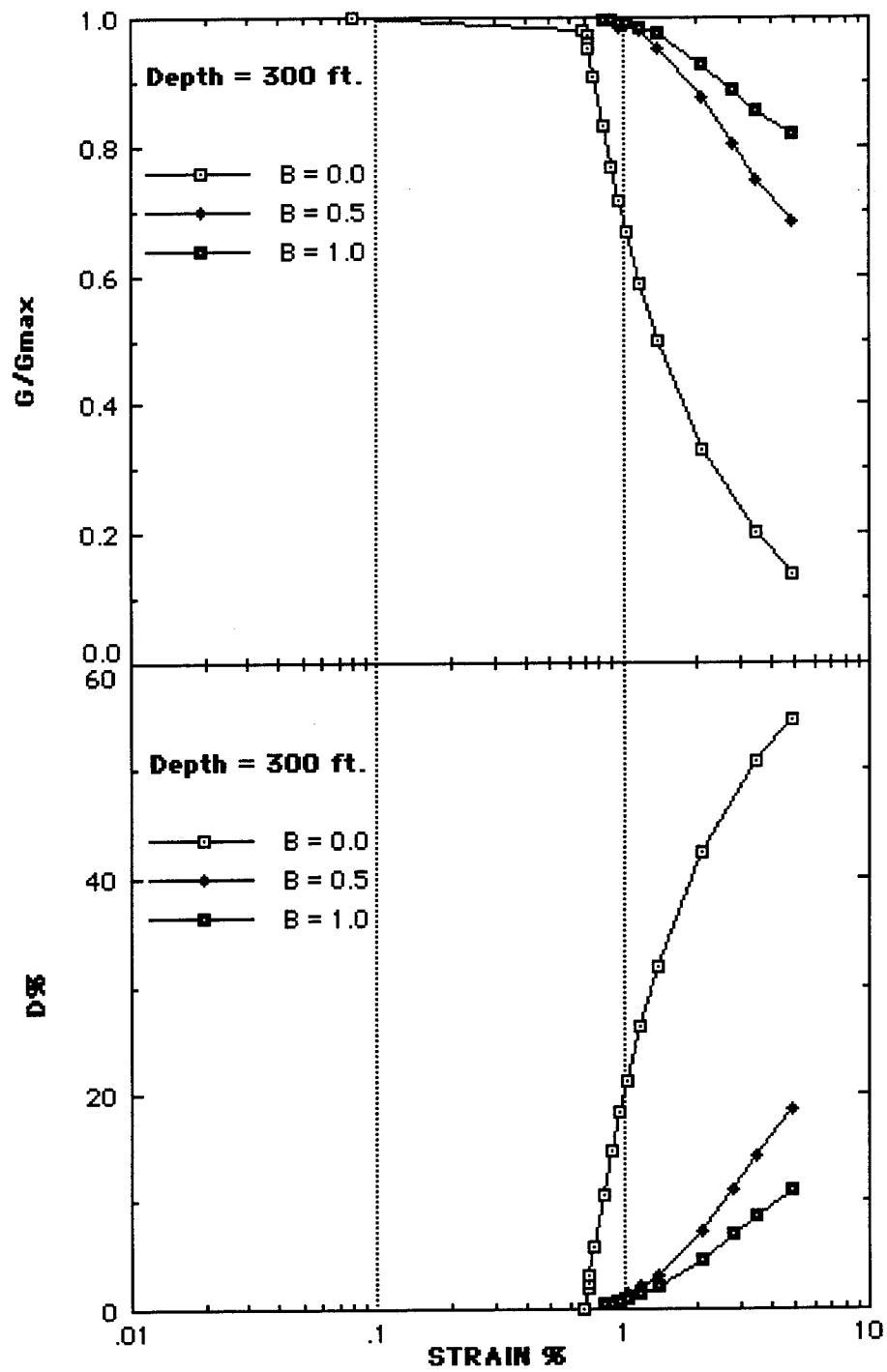


Figure 6.D-6
Cyclic behavior of Coulomb-Mohr material at deep depth

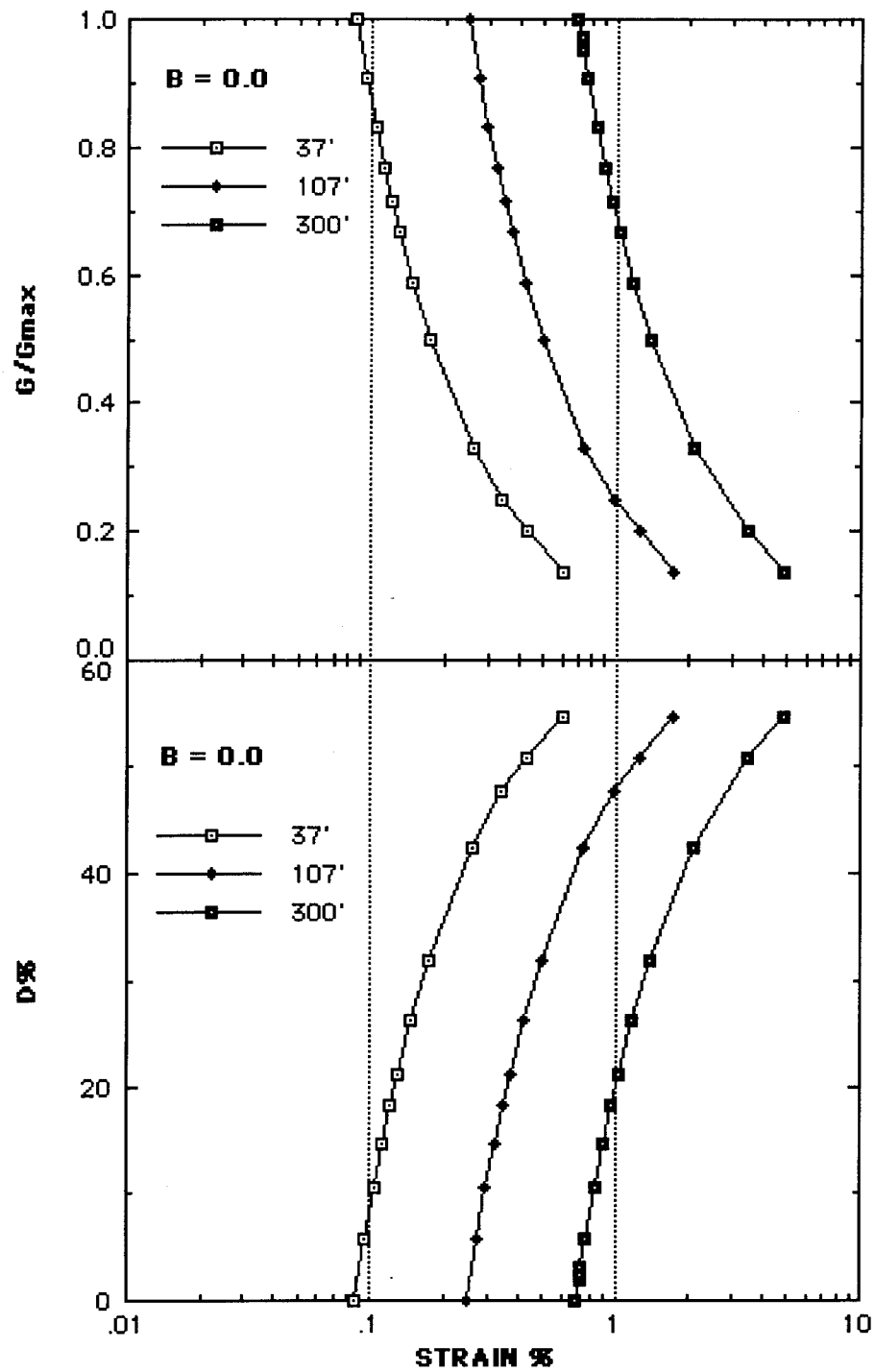


Figure 6.D-7
Comparison of depth effects on degradation data for Coulomb-Mohr material with complete nonassociated flow rule

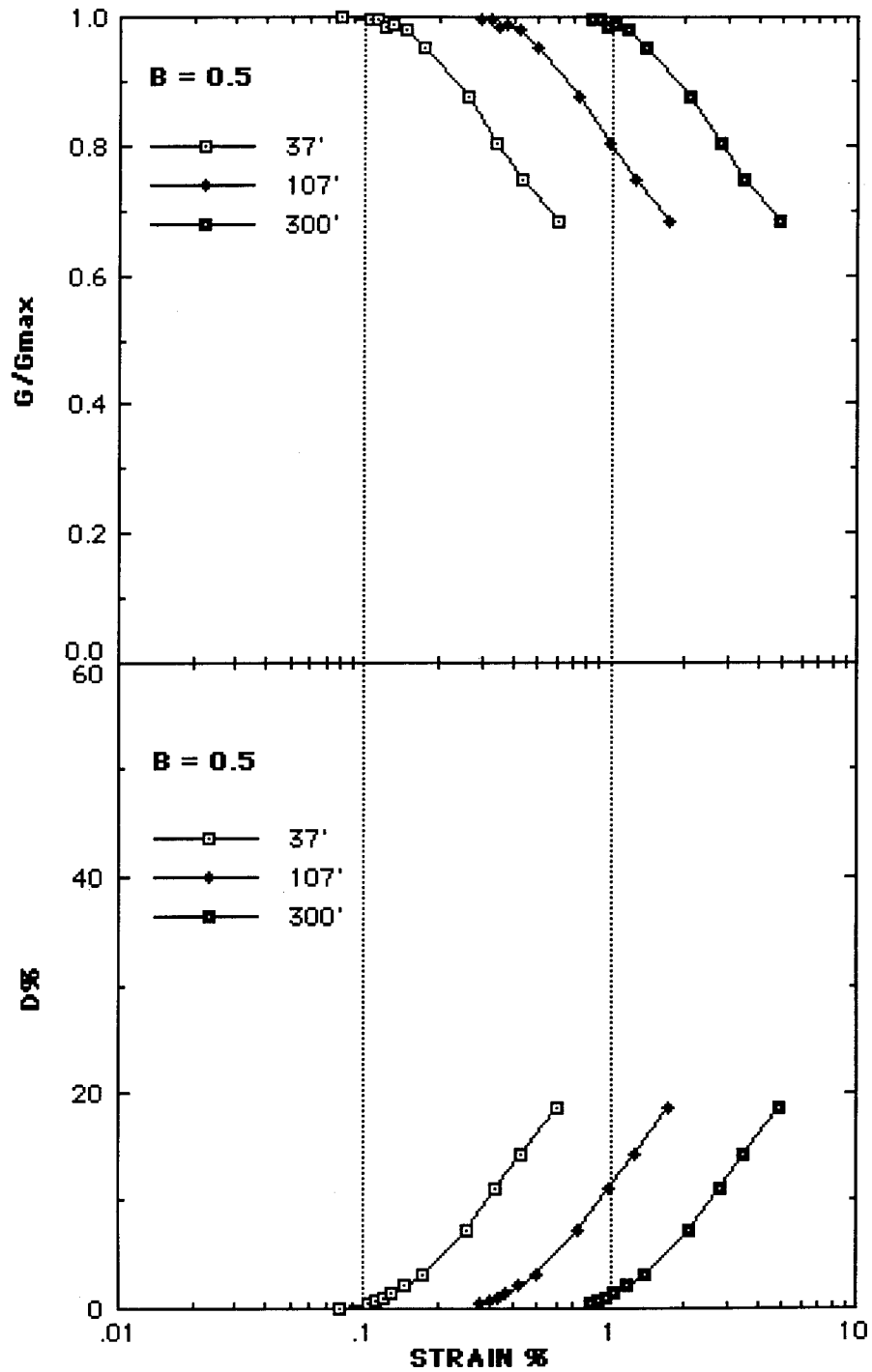


Figure 6.D-8
Comparison of depth effects on degradation data for Coulomb-Mohr material with partial nonassociated flow rule

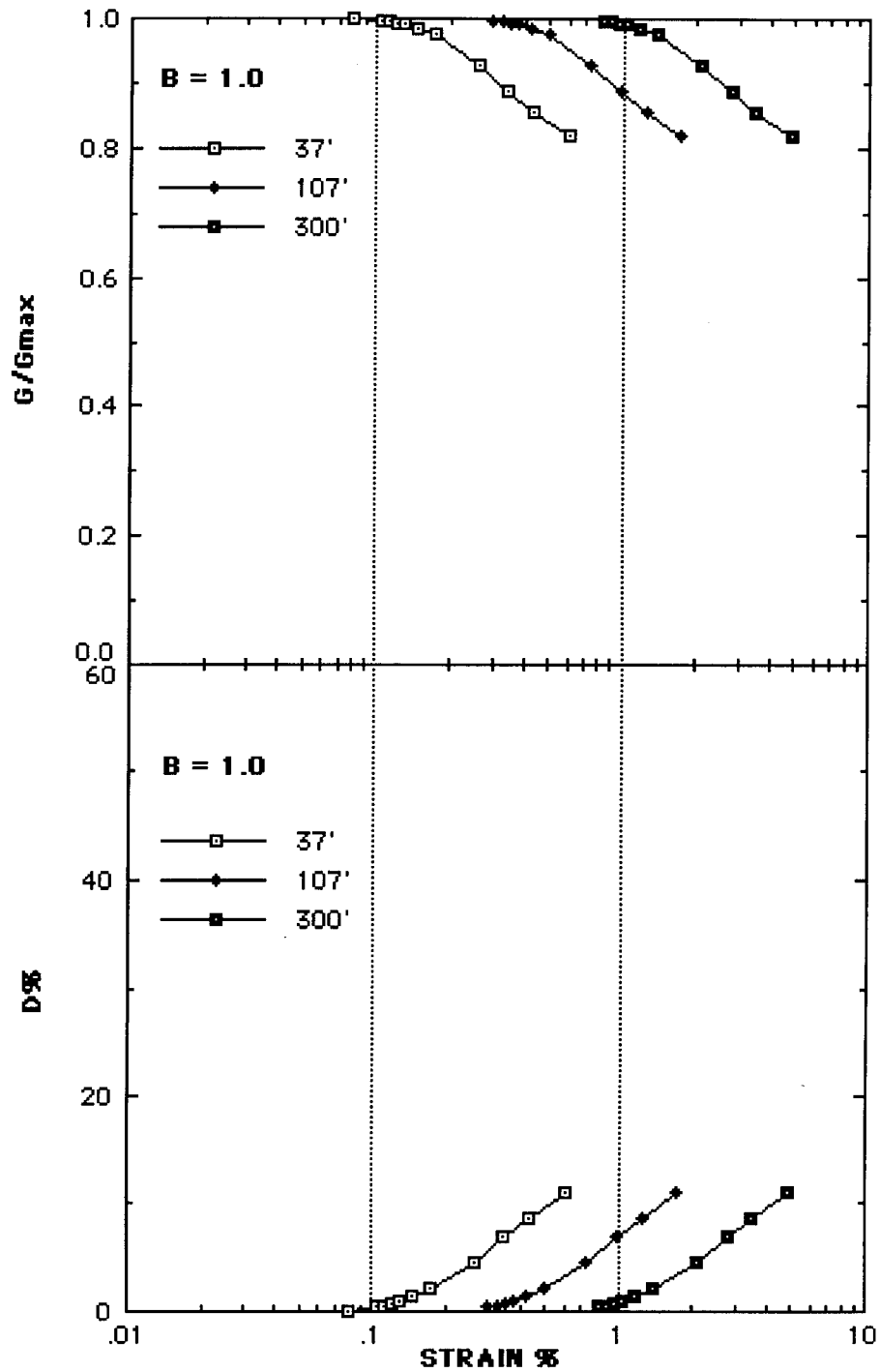


Figure 6.D-9
Comparison of depth effects on degradation data for Coulomb-Mohr material with full associated flow rule

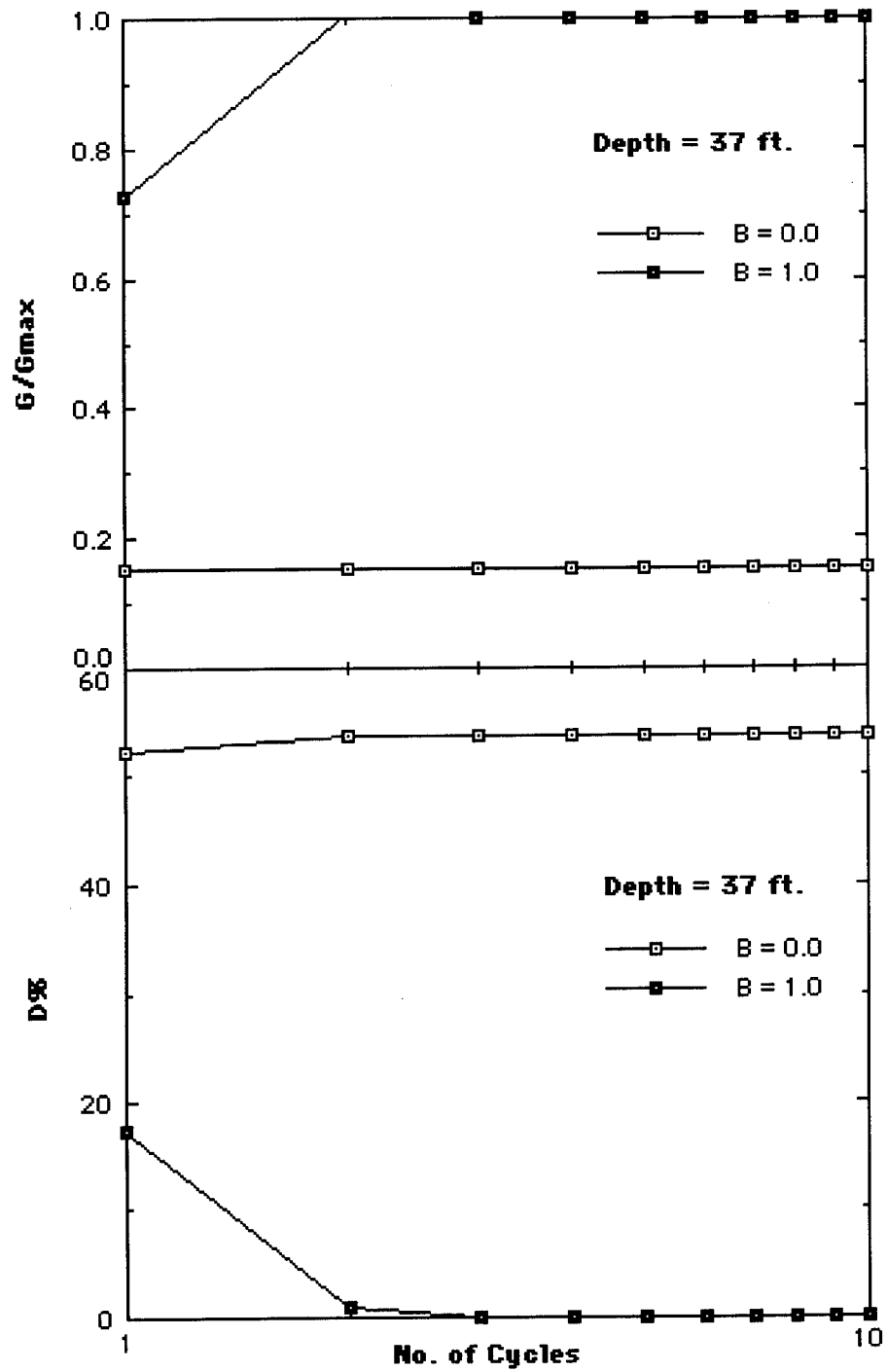


Figure 6.D-10
Comparison of compacting models for shallow depth, max. shear strain=3.1%

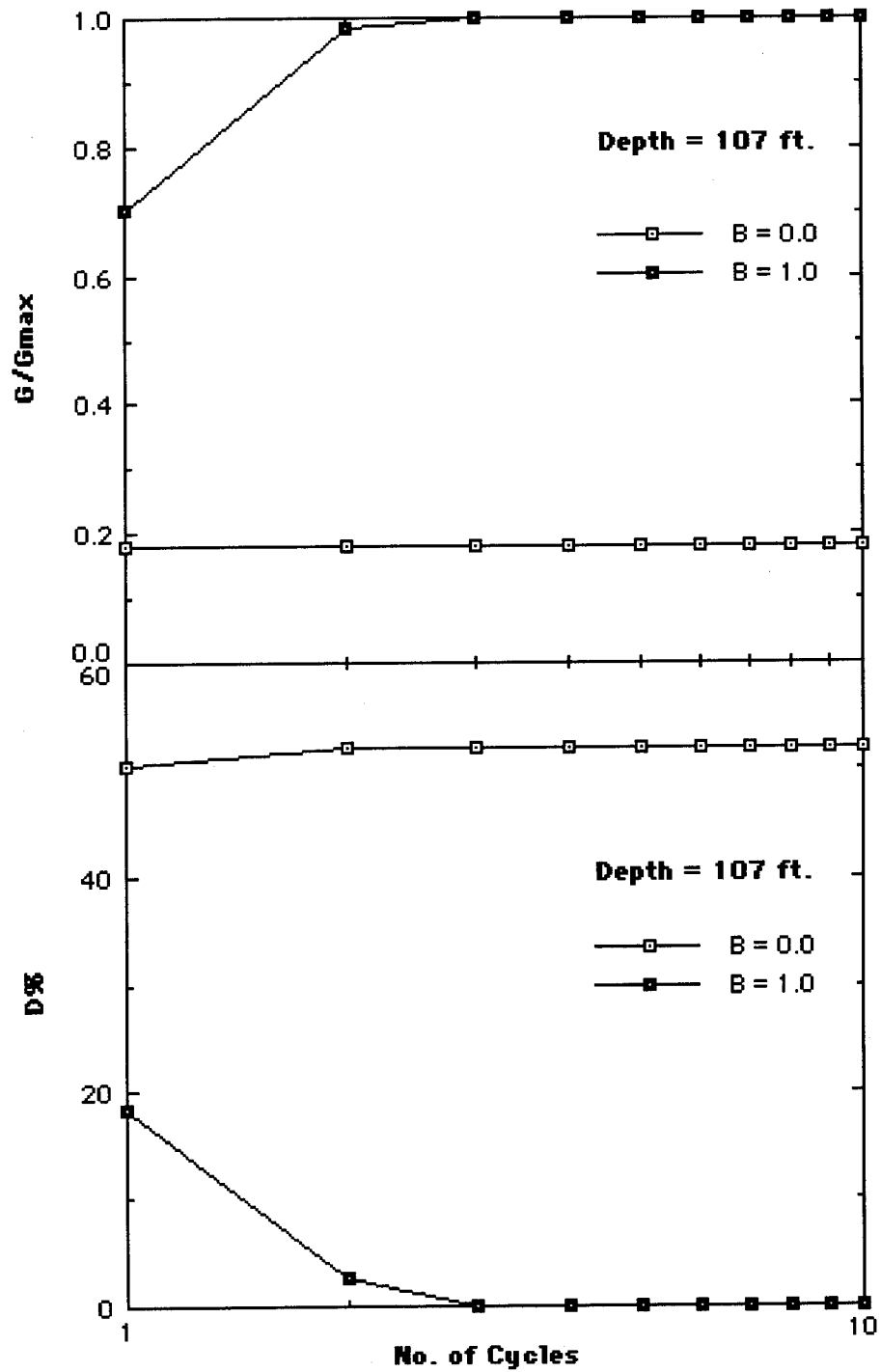


Figure 6.D-11
Comparison of compacting models at intermediate depth, max. shear strain=4.75%

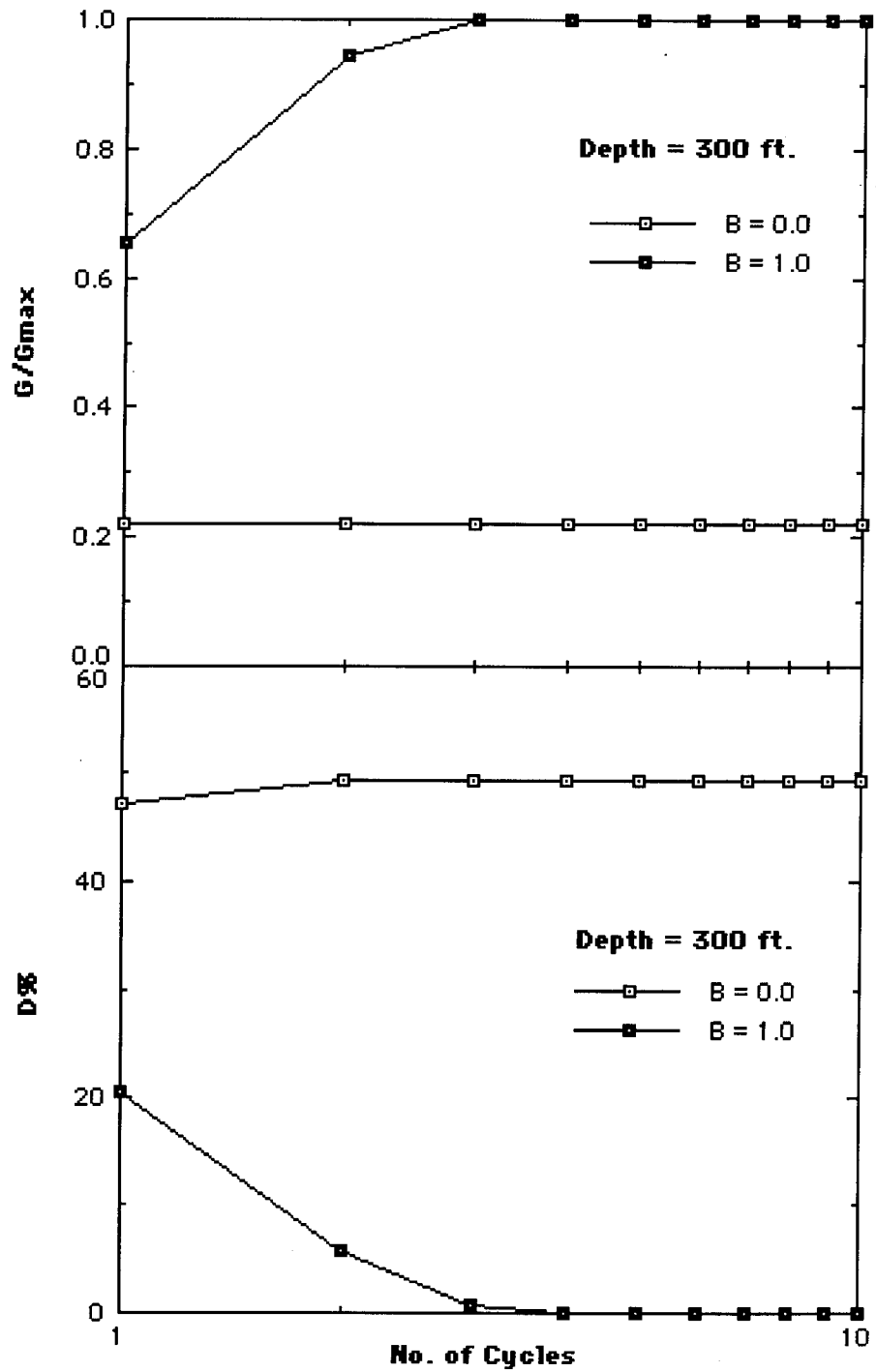


Figure 6.D-12
Comparison of compacting model at deep depth, max. shear strain=8.4%

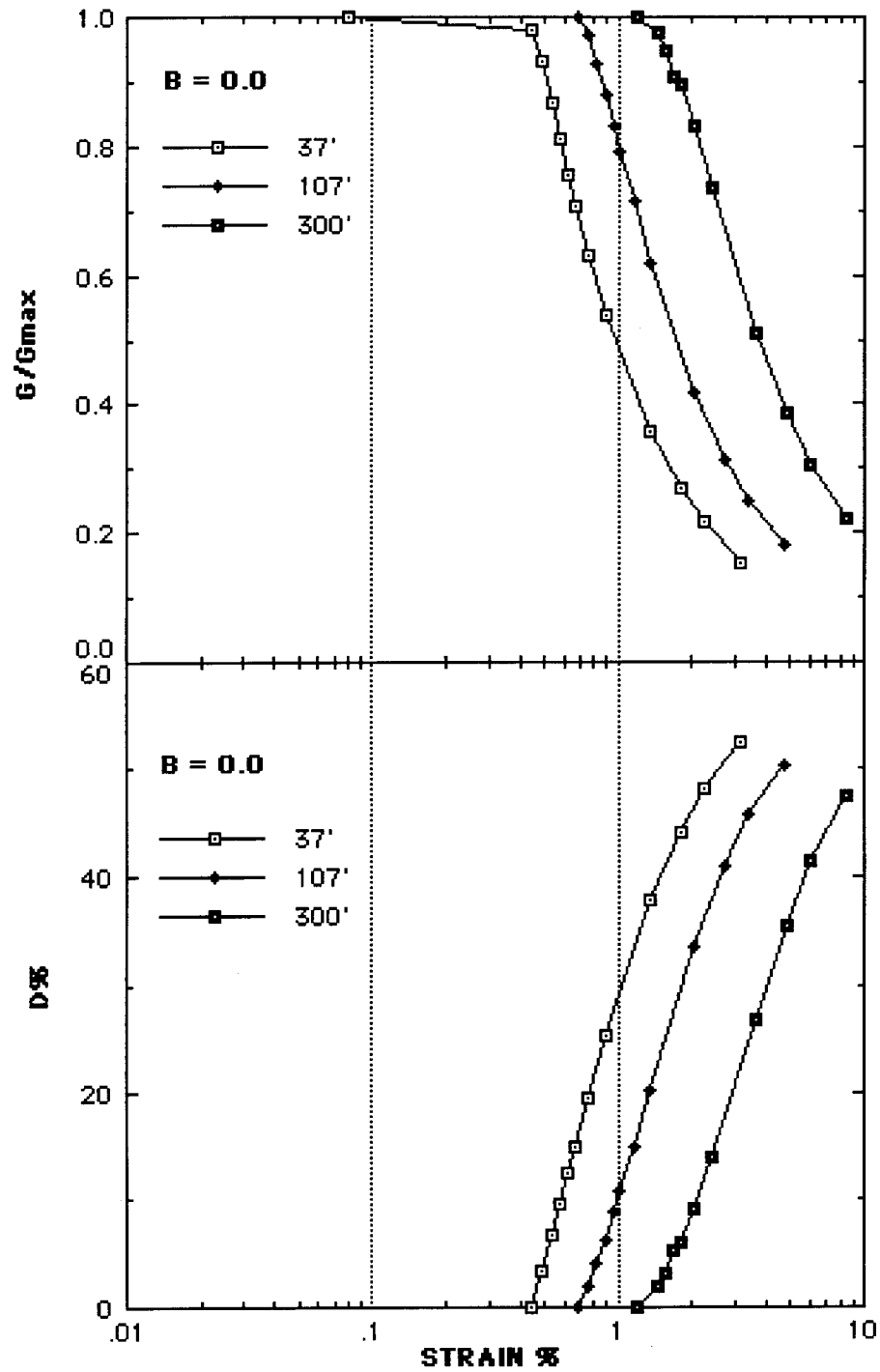


Figure 6.D-13
Cyclic behavior of compacting model with nonassociated flow rule

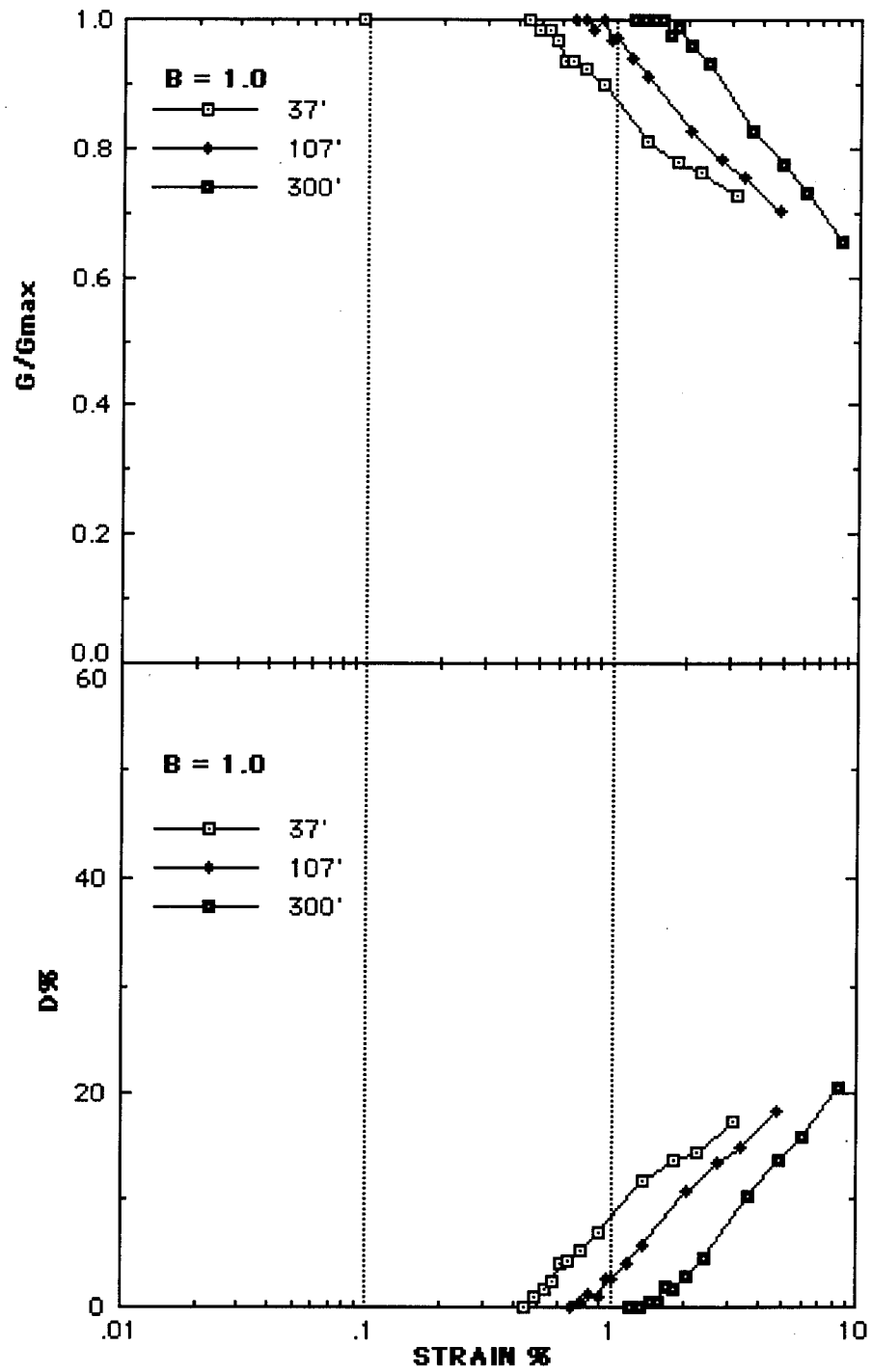


Figure 6.D-14
Cyclic behavior of compacting model with associated flow rule

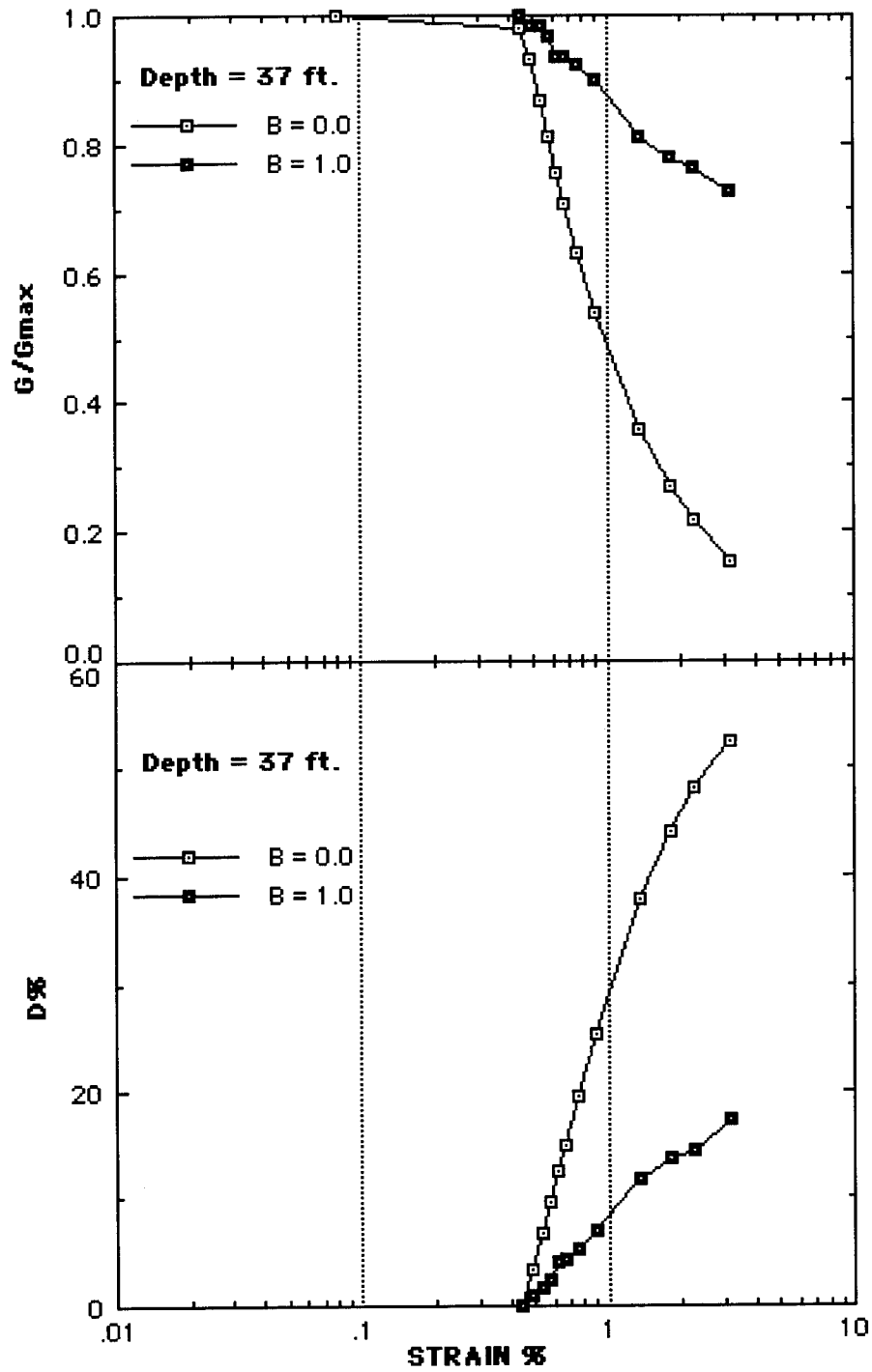


Figure 6.D-15
Cyclic behavior of compacting model at shallow depth

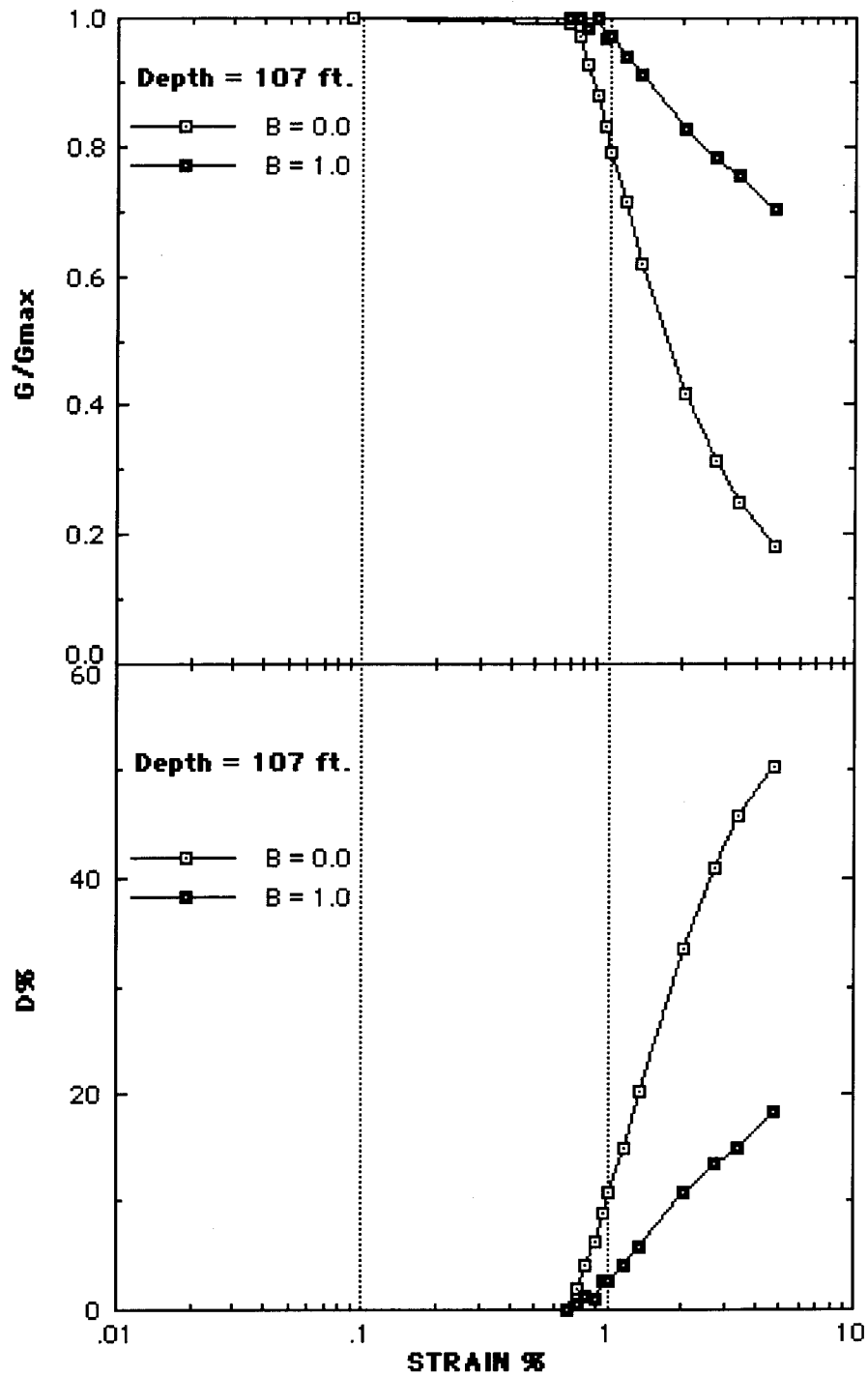


Figure 6.D-16
Cyclic behavior of compacting model at intermediate depth

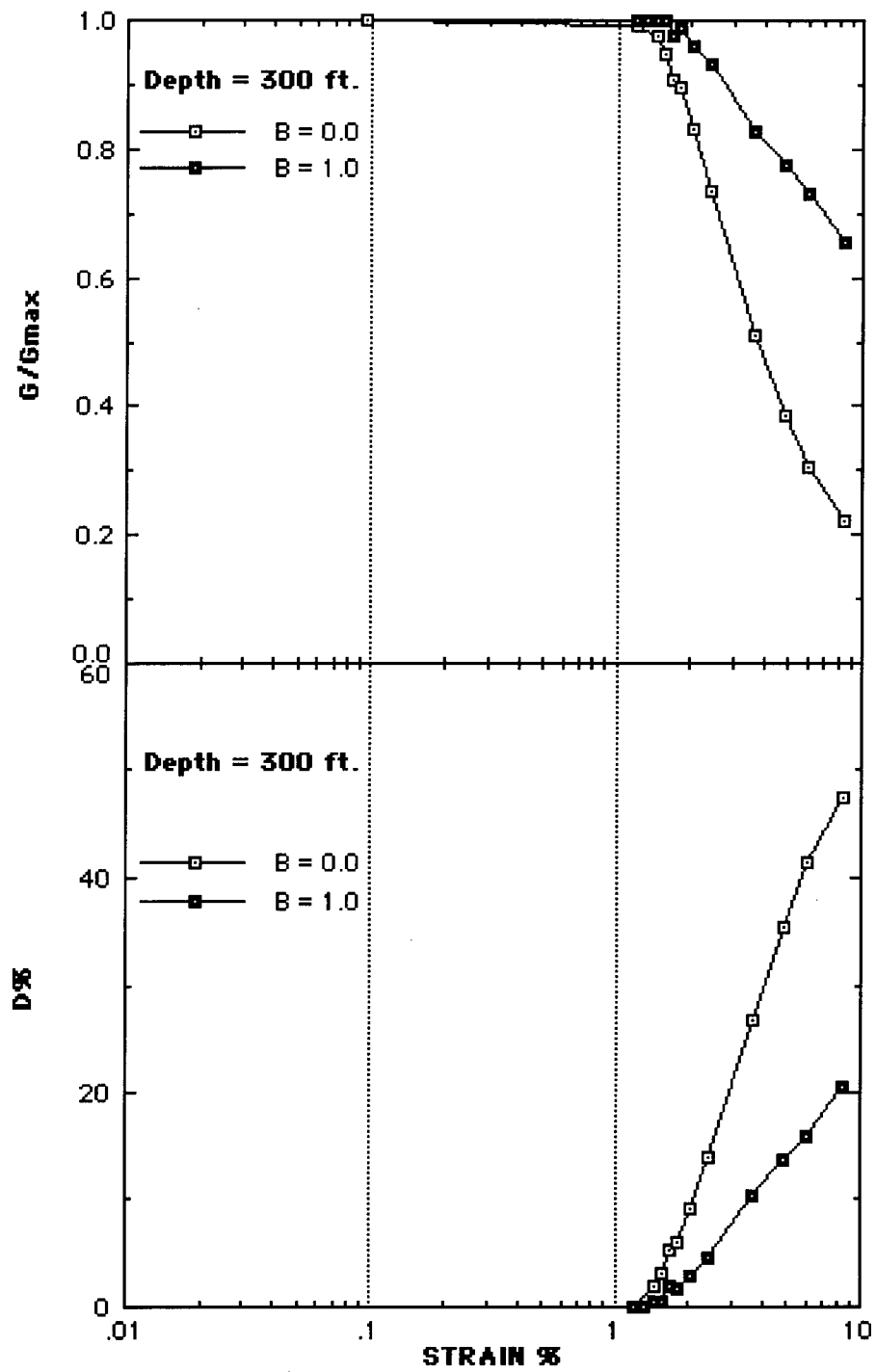


Figure 6.D-17
Cyclic behavior of compacting model at deep depth

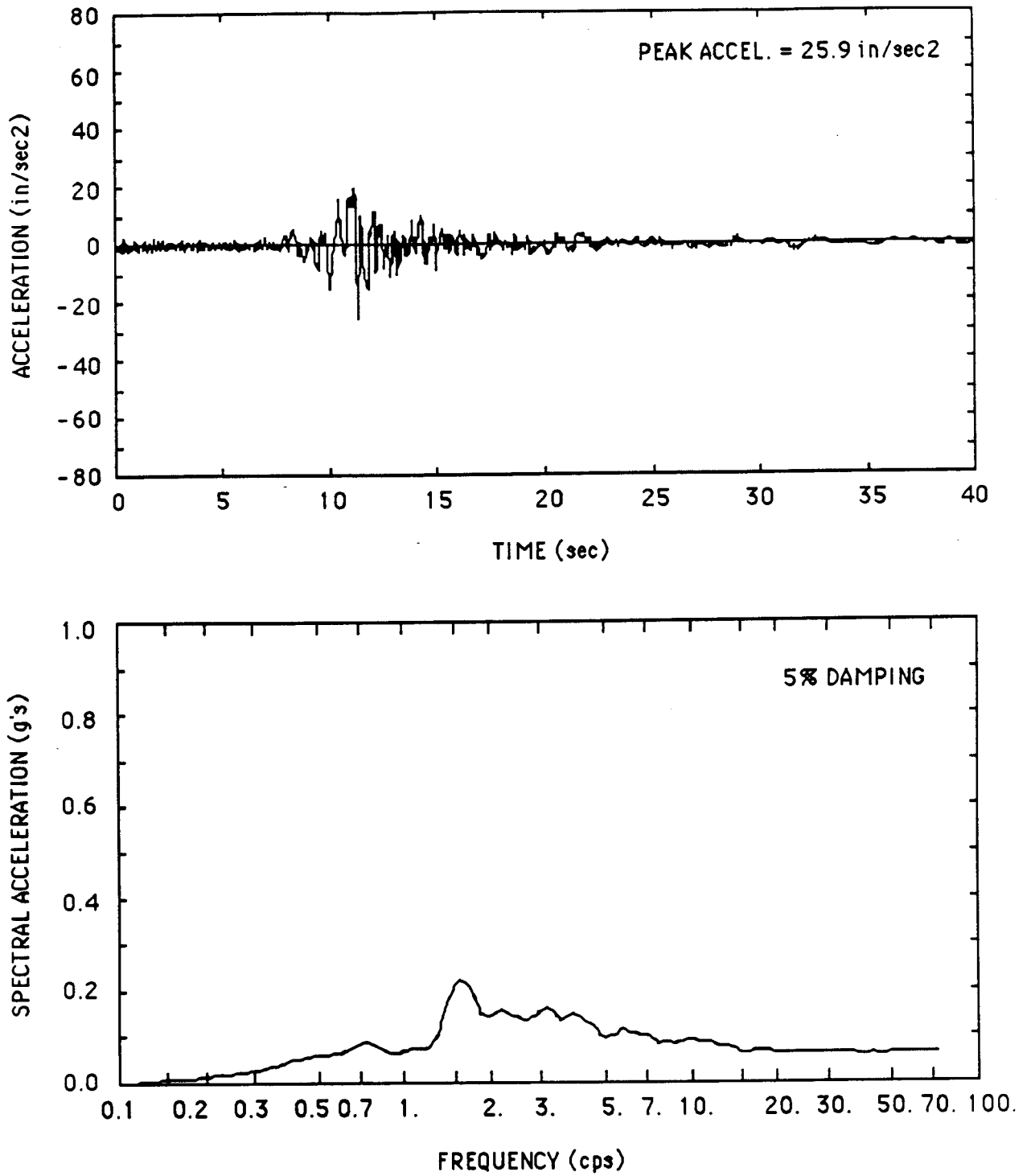


Figure 6.D-18
Yerba Buena Island, input motion: measured horizontal motion (channel 1)

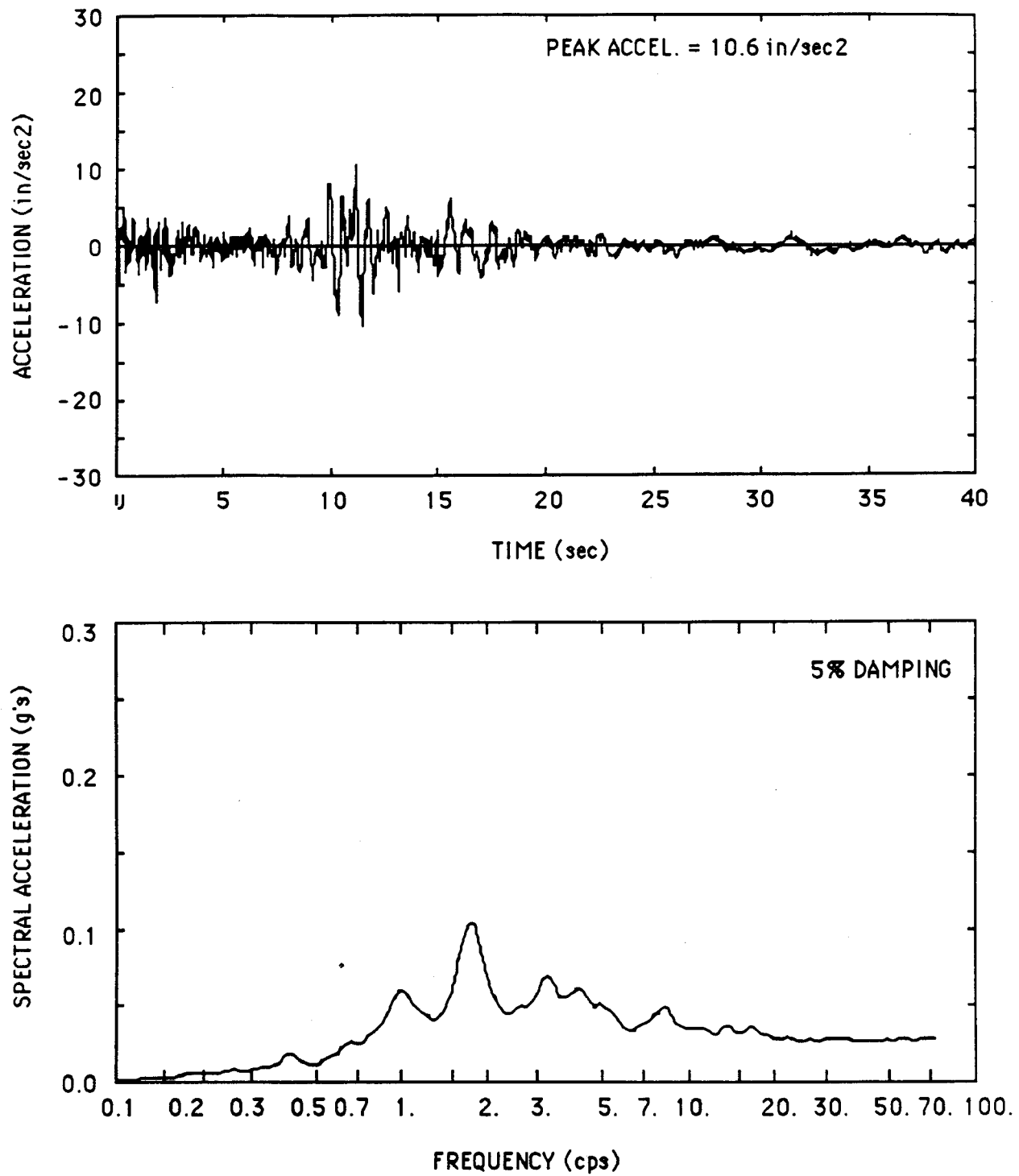


Figure 6.D-19
Yerba Buena Island, input motion: measured vertical motion (channel 2)

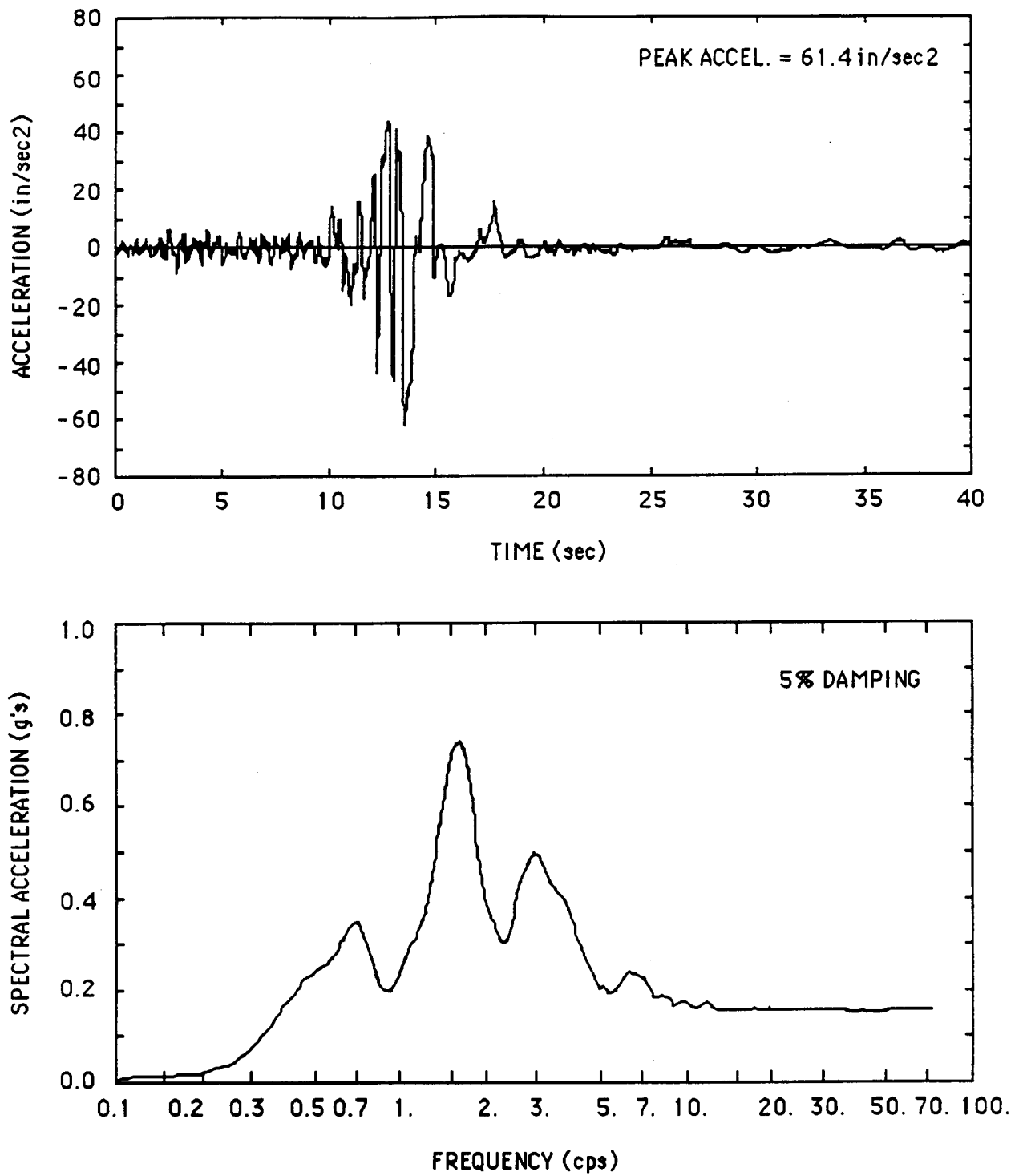


Figure 6.D-20
Treasure Island soil column: measured horizontal motion (channel 1)

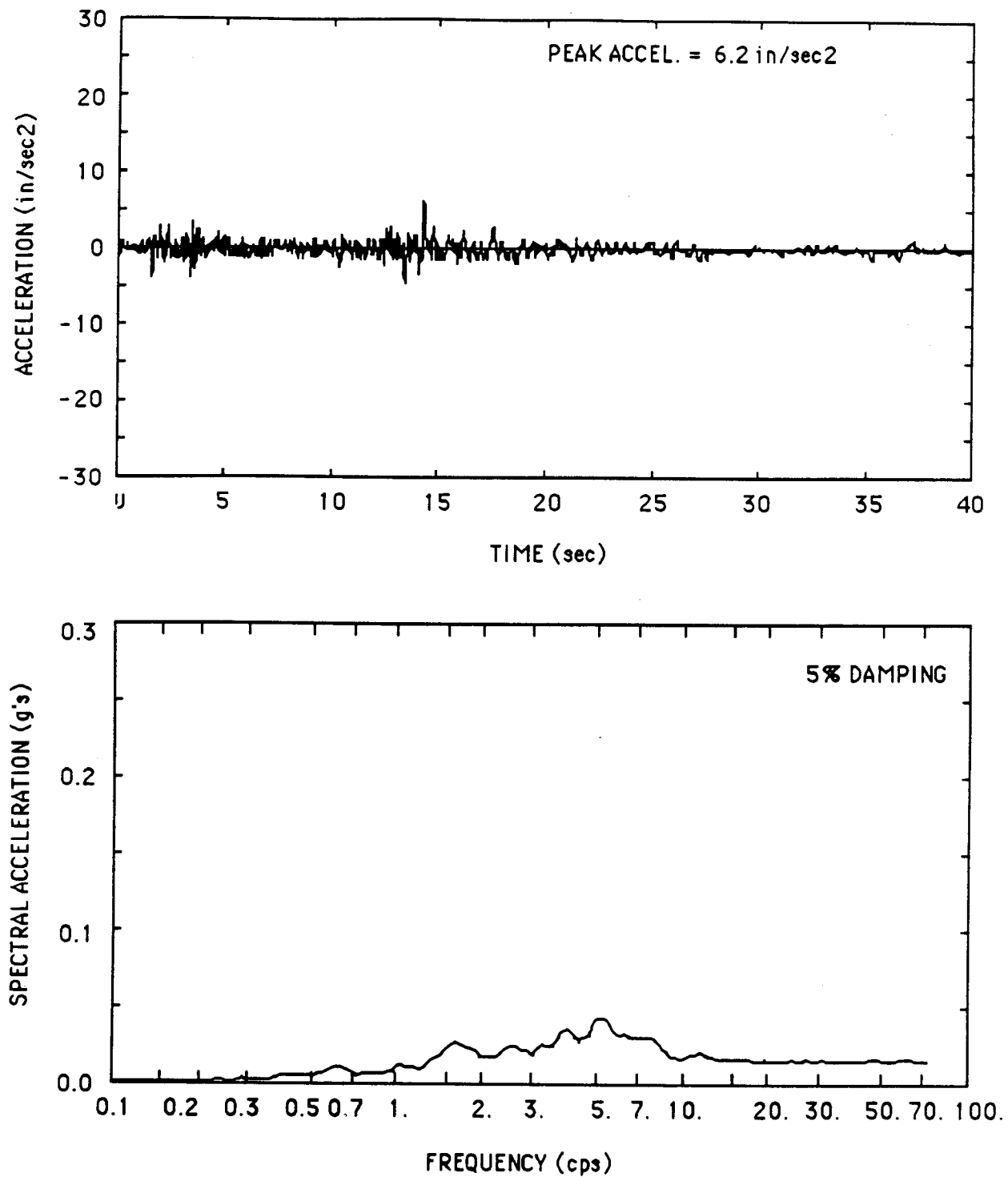


Figure 6.D-21
Treasure Island soil column: measured vertical motion (channel 2)

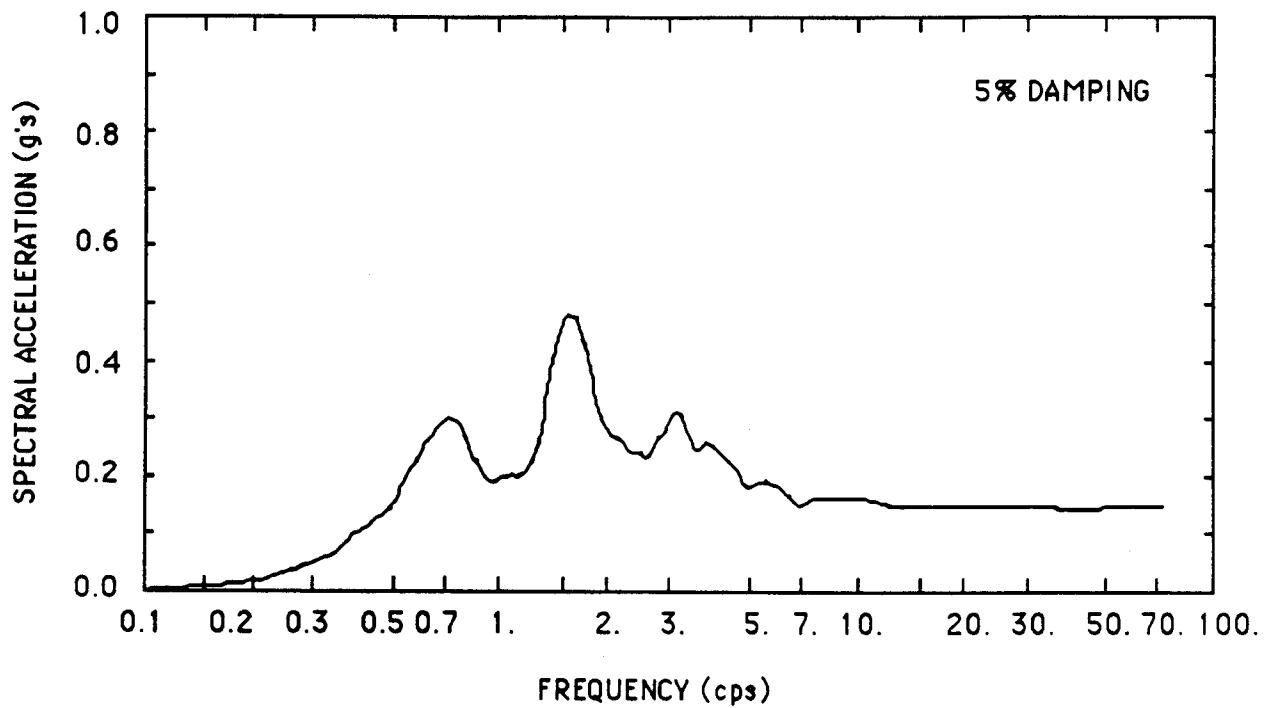
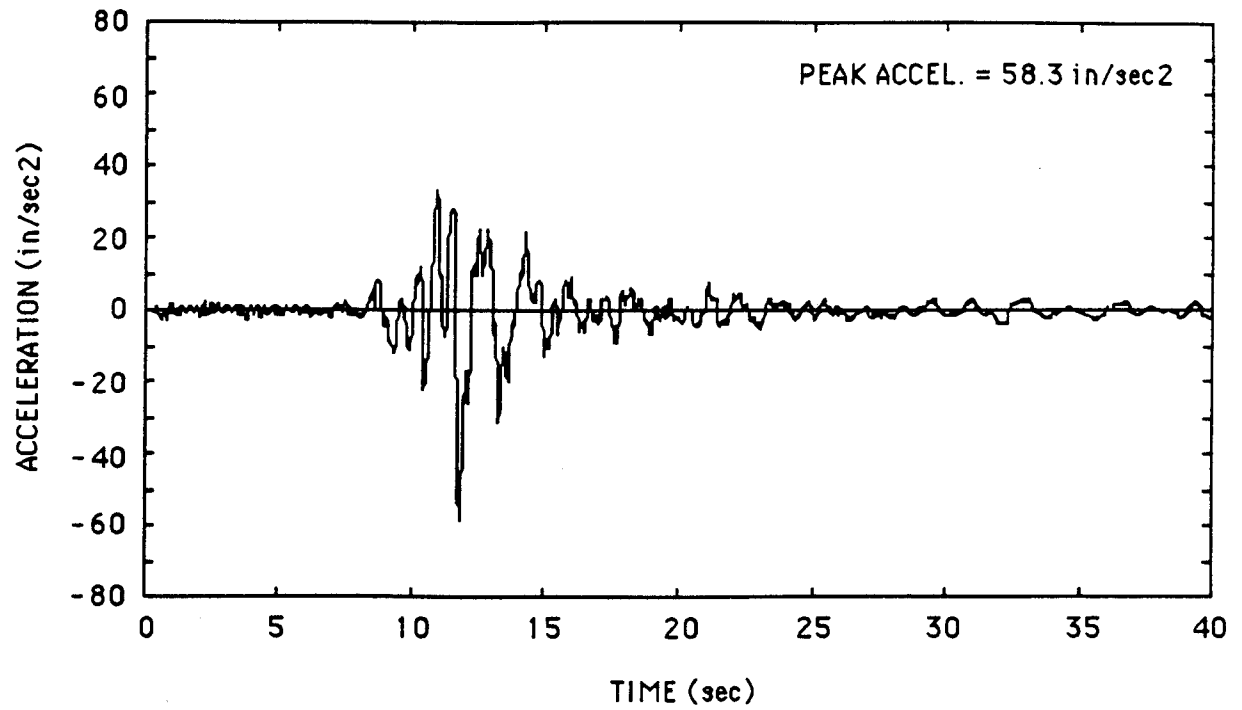


Figure 6.D-22

Treasure Island soil column: horizontal surface motion using YBI channel 1 input with iterated linear convolution and Stokoe soil degradation properties

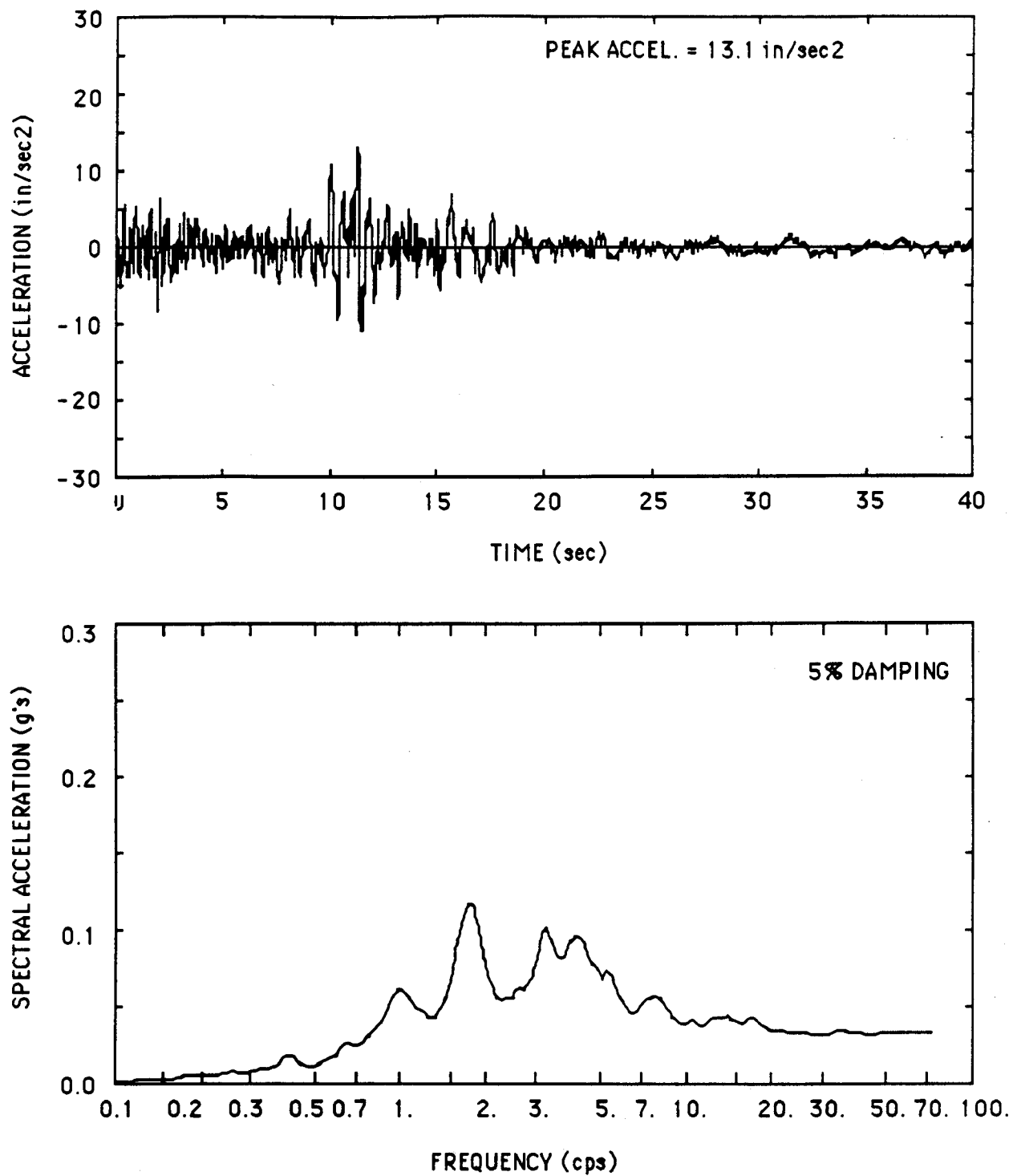


Figure 6.D-23

Treasure Island soil column: vertical motion using YBI channel 2 input with linear convolution, no soil degradation and 2% hysteretic damping ratio

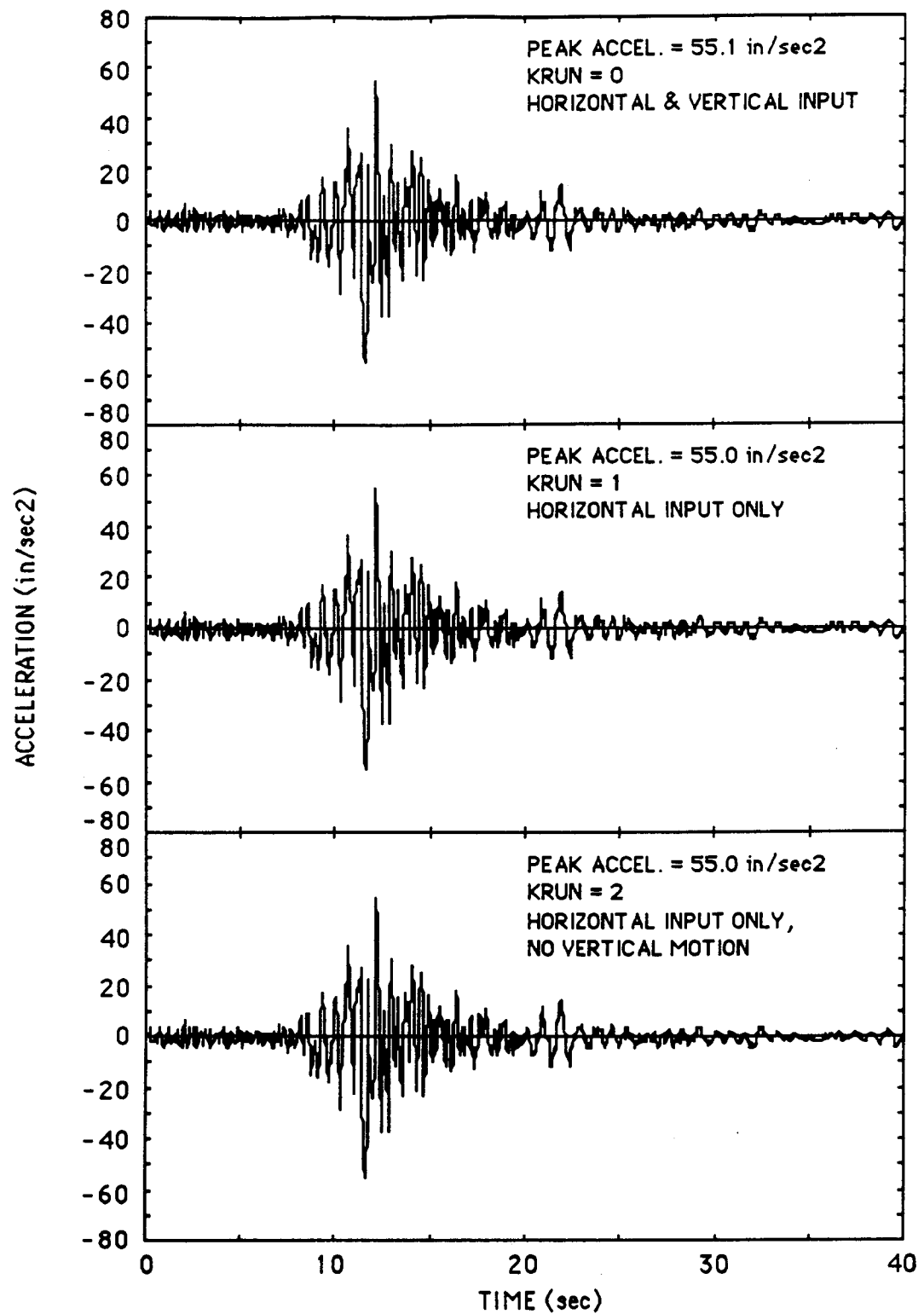


Figure 6.D-24

Treasure Island soil column: horizontal motion for elastic soil model from time domain calculation (max poisson's ratio = 0.45)

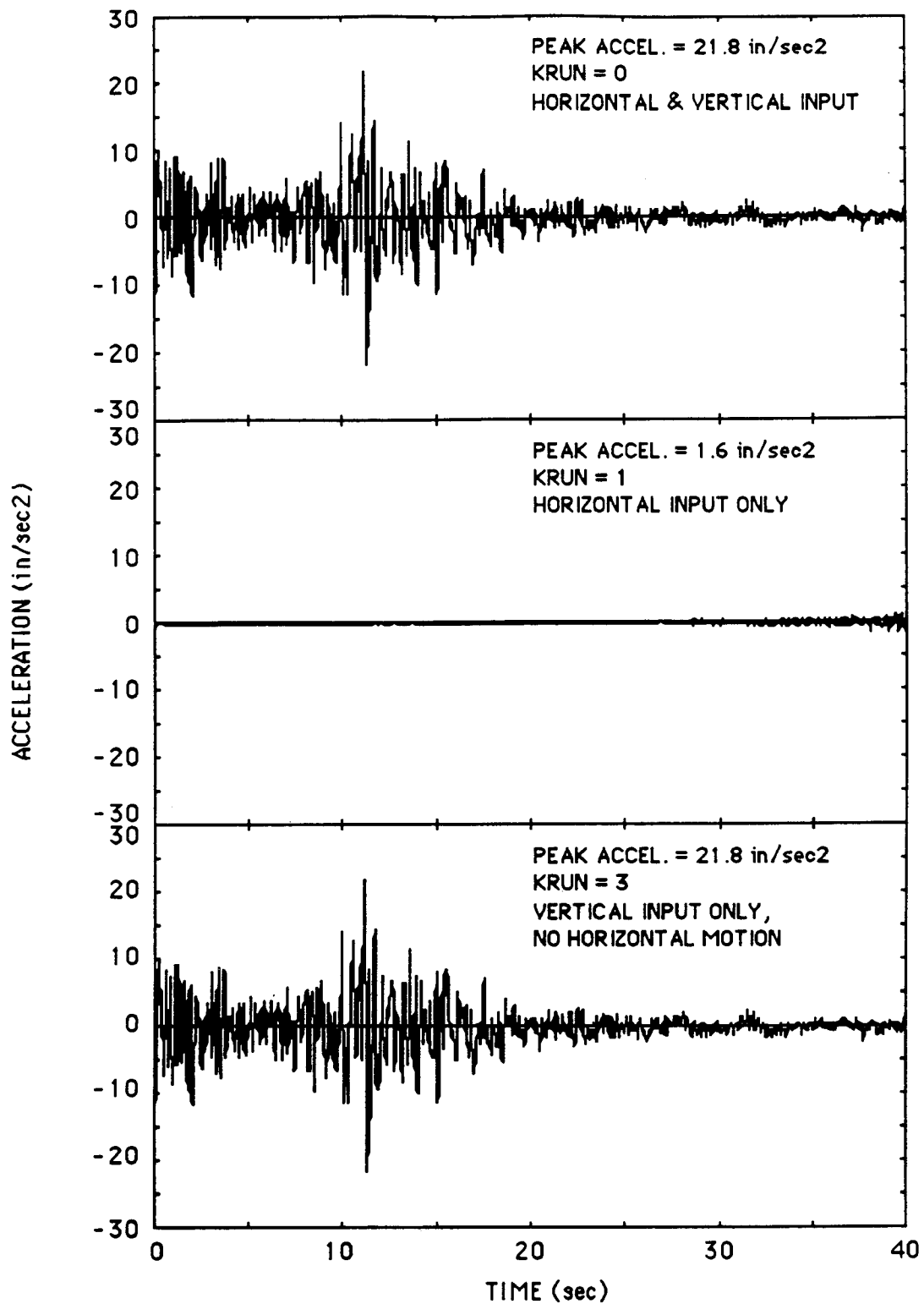


Figure 6.D-25

Treasure Island soil column: vertical motion for elastic soil model from time domain calculation (max poisson's ratio = 0.45)

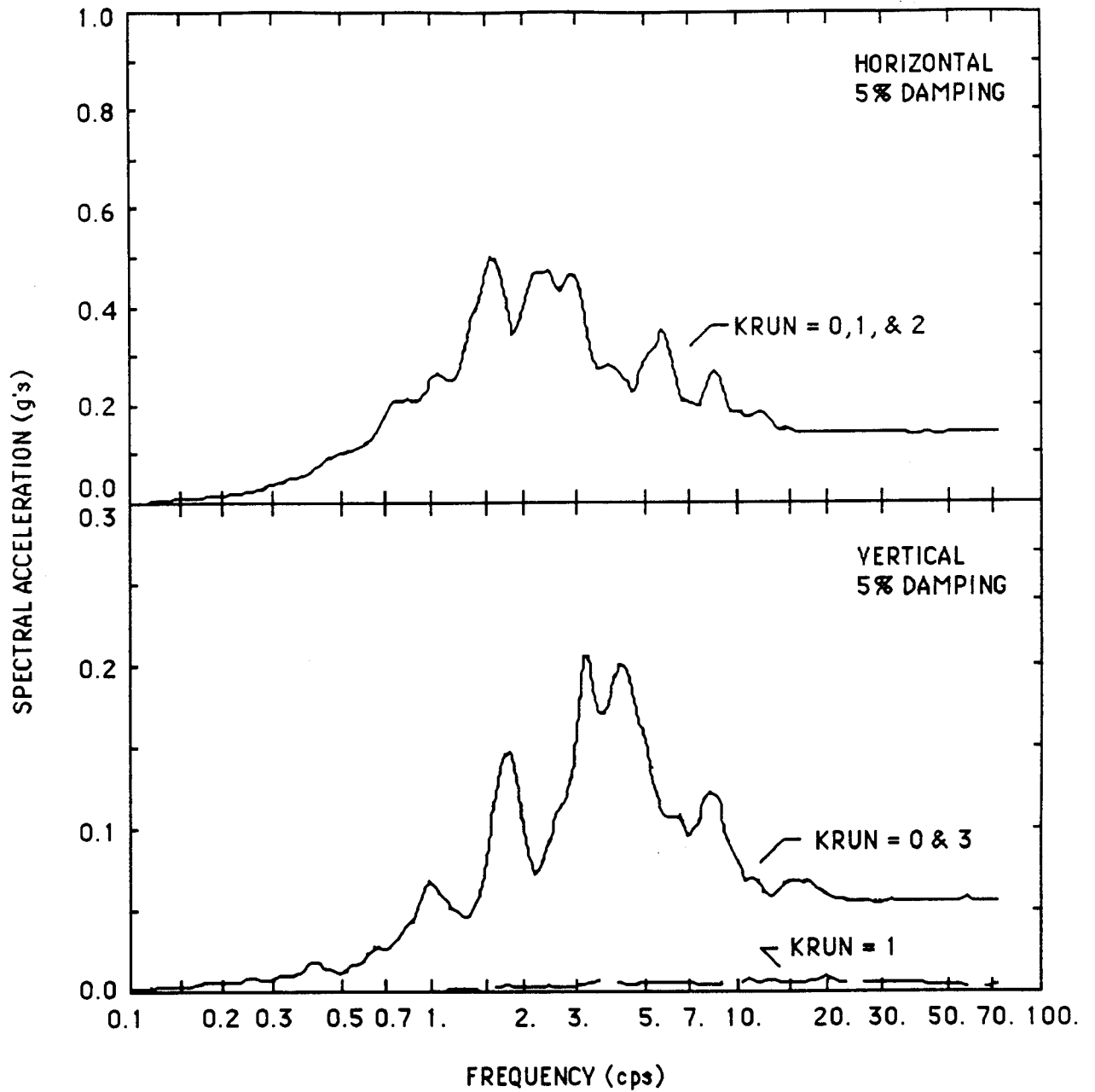


Figure 6.D-26
Treasure Island soil column: spectra for elastic soil column from time domain calculation
(max poisson's ratio = 0.45)

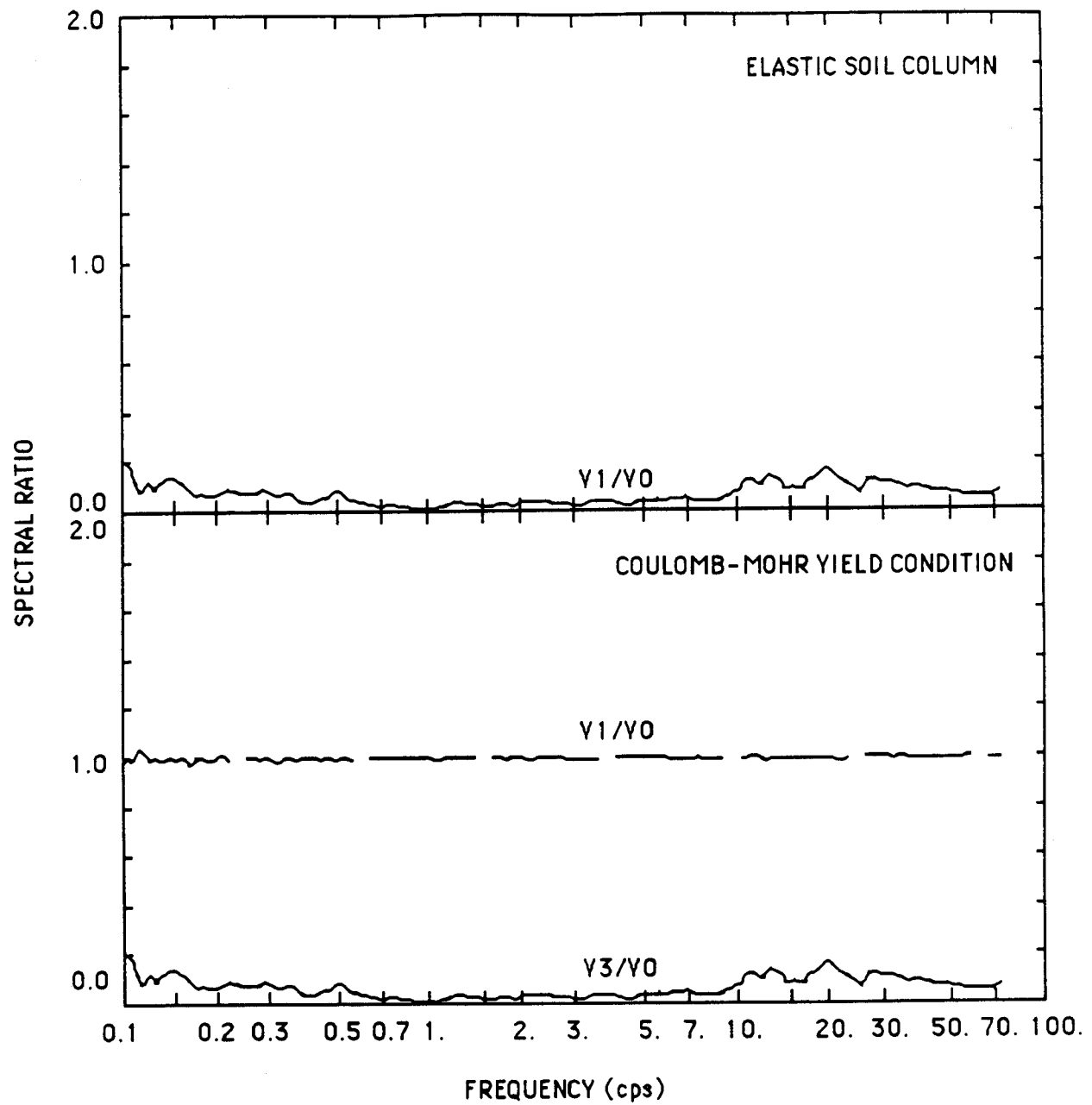


Figure 6.D-27
 Treasure Island soil column: vertical spectral ratio from time domain calculation
 (poisson's ratio = 0.45)

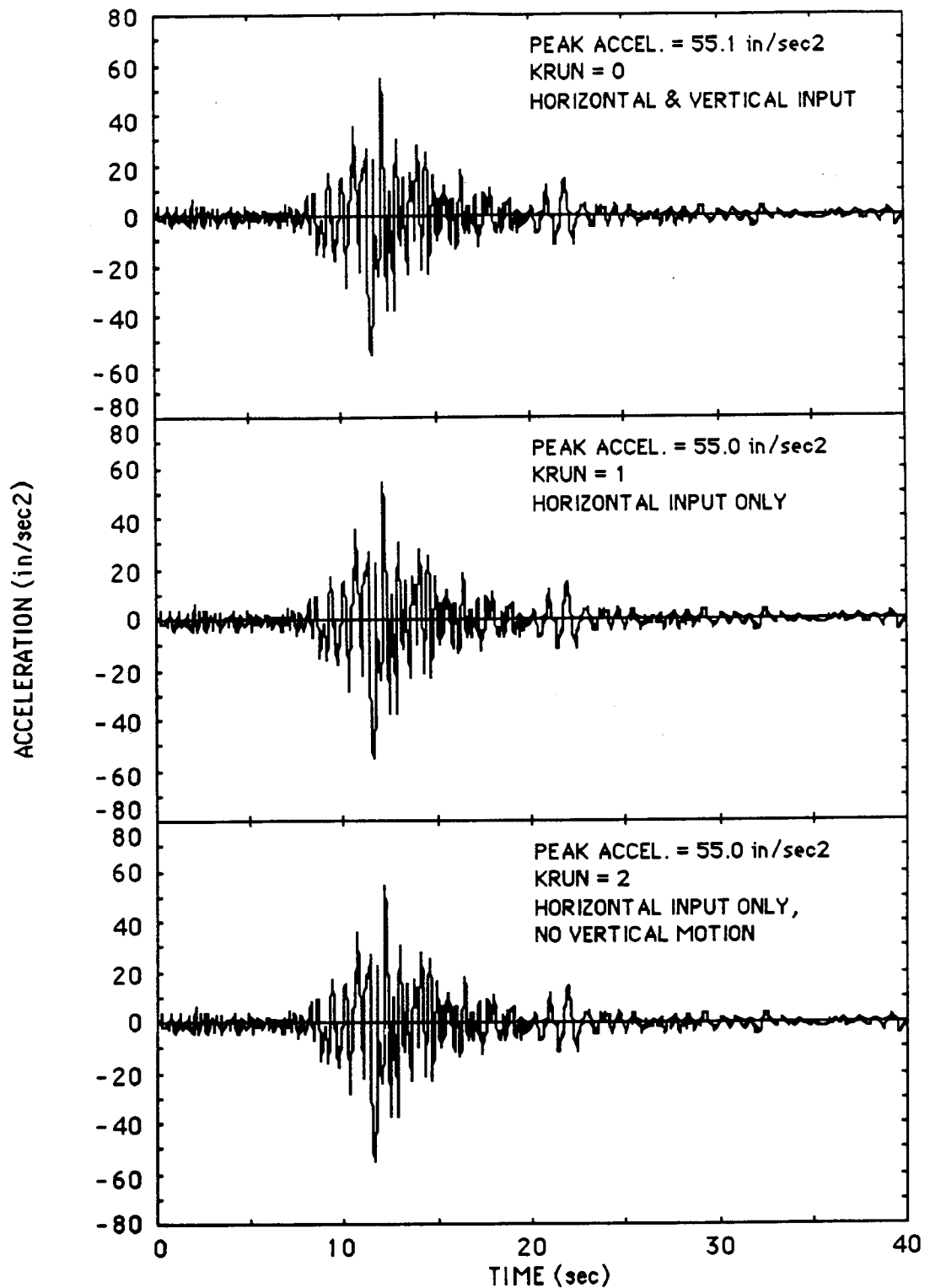


Figure 6.D-28

Treasure Island soil column: horizontal motion for Coulomb-Mohr soil model from time domain calculation (max poisson's ratio = 0.45)

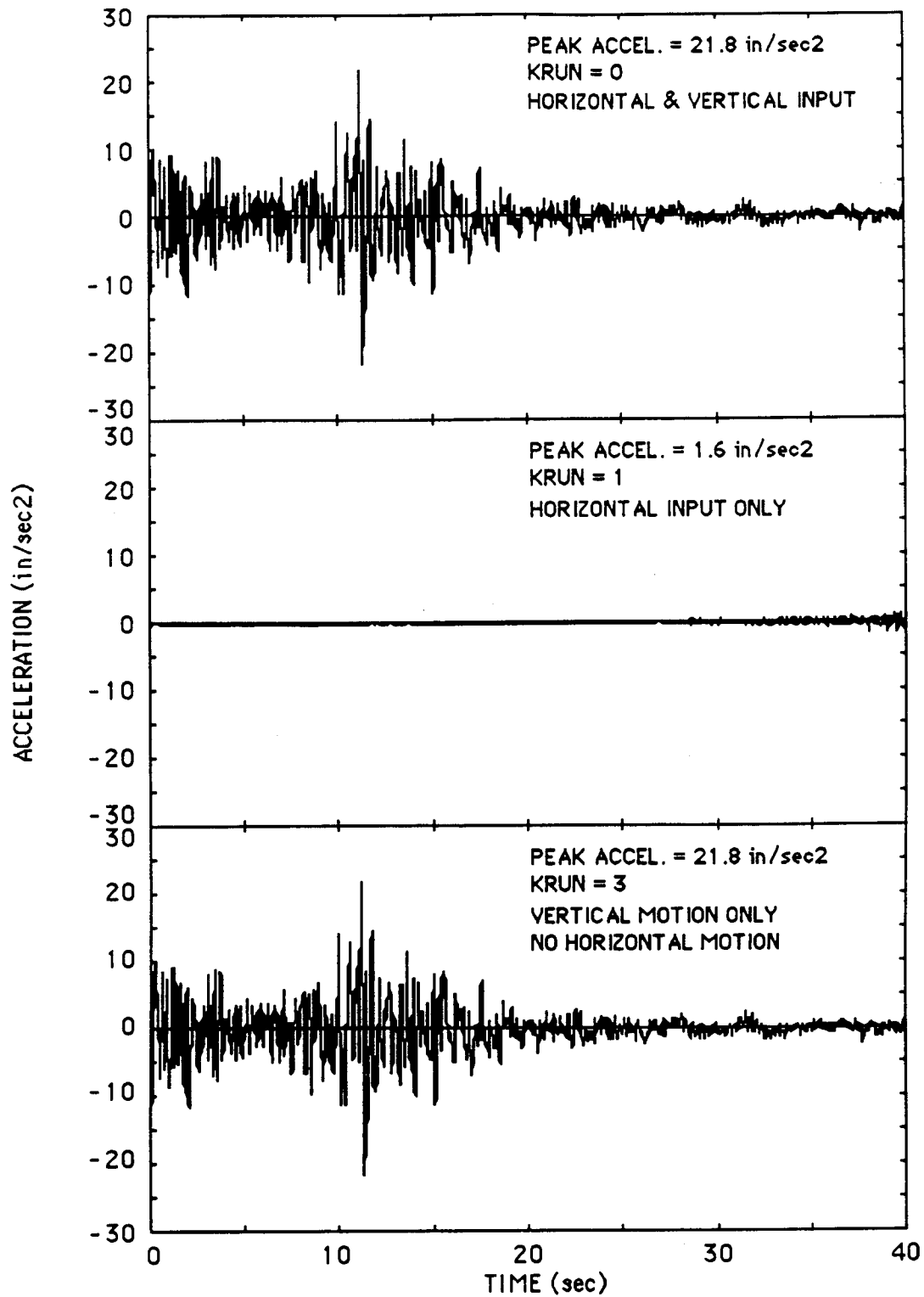


Figure 6.D-29

Treasure Island soil column: vertical motion for Coulomb-Mohr soil model from time domain calculation (max poisson's ratio = 0.45)

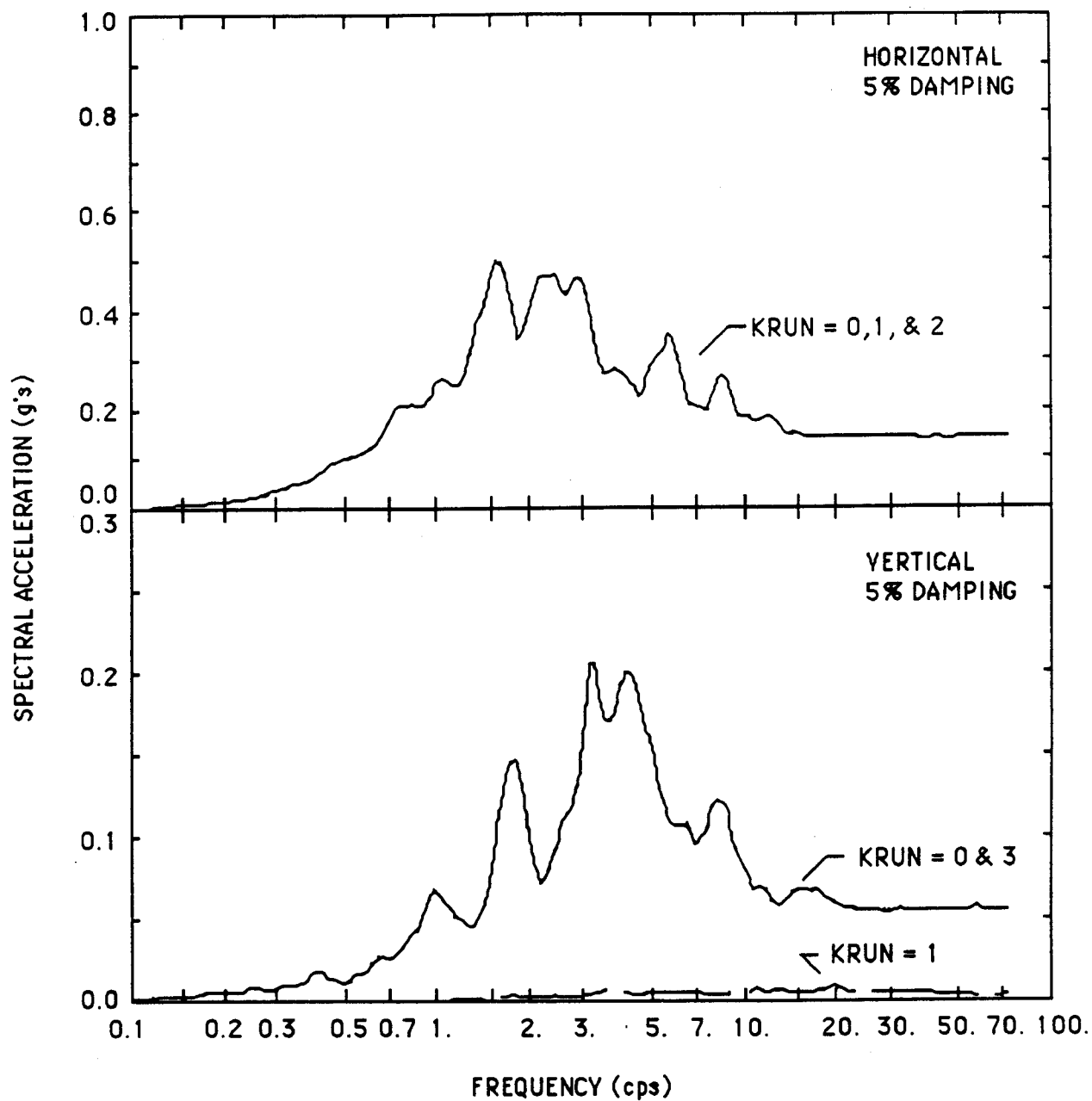


Figure 6.D-30
 Treasure Island soil column: response spectra for Coulomb-Mohr soil model from time domain calculation (max poisson's ratio = 0.45)

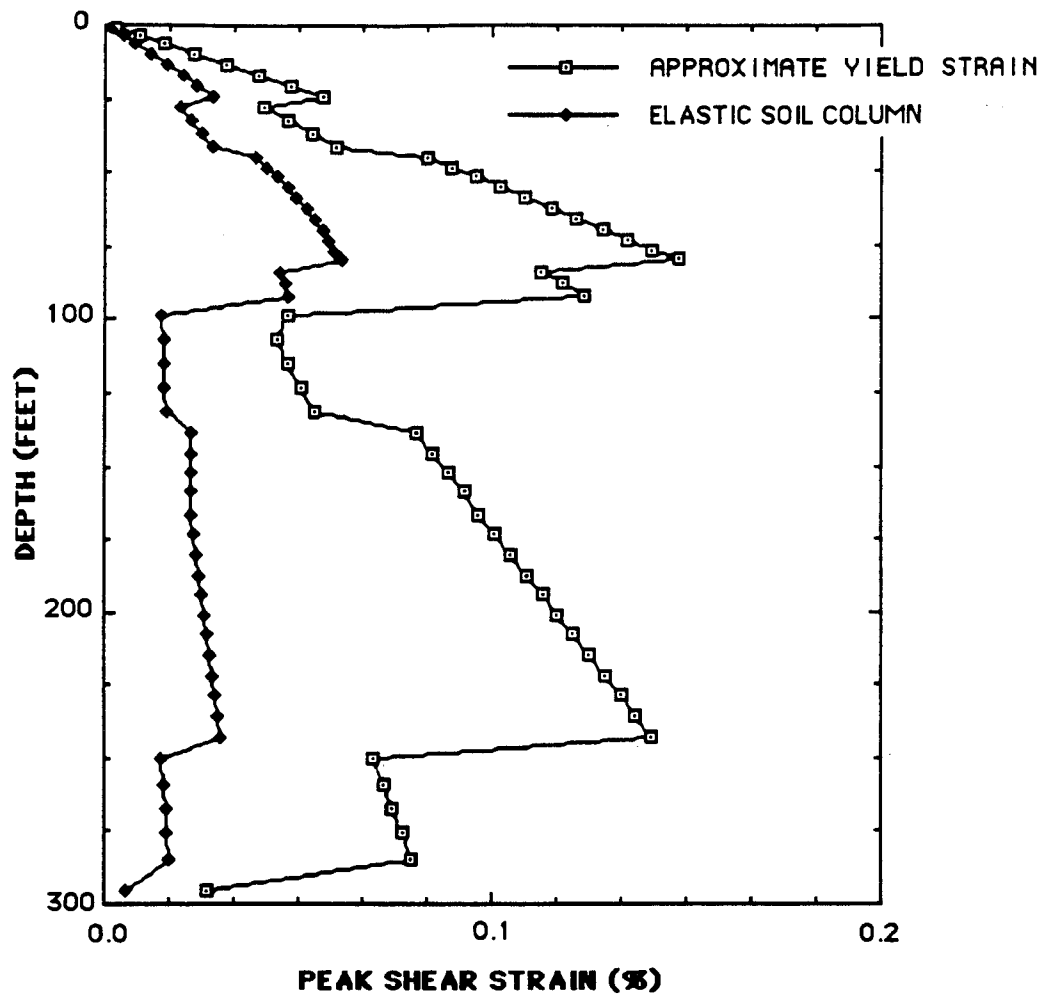


Figure 6.D-31
Peak shear strains for treasure Island from elastic soil column

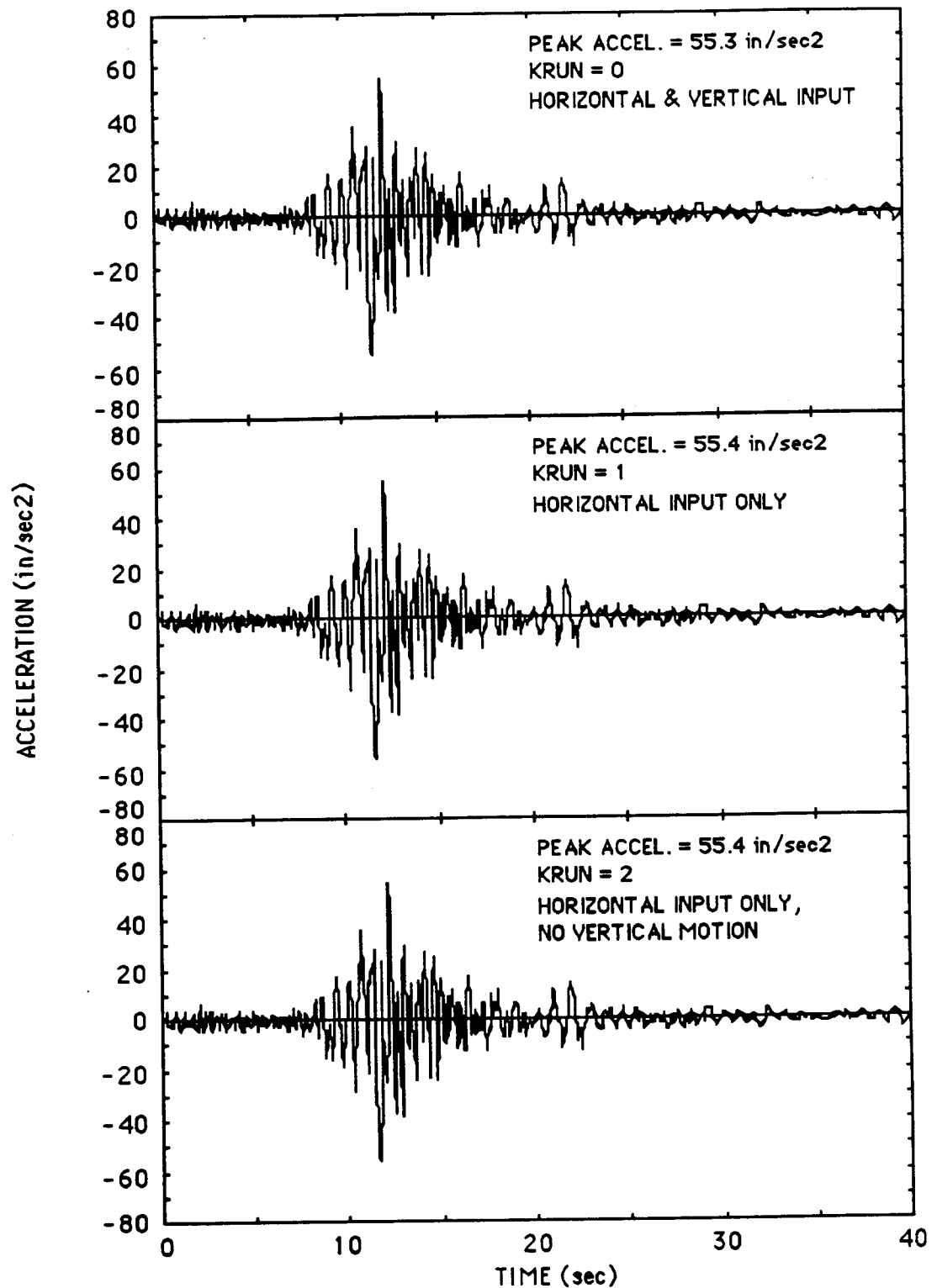


Figure 6.D-32

Treasure Island soil column: horizontal motion for compacted soil model from time domain calculation

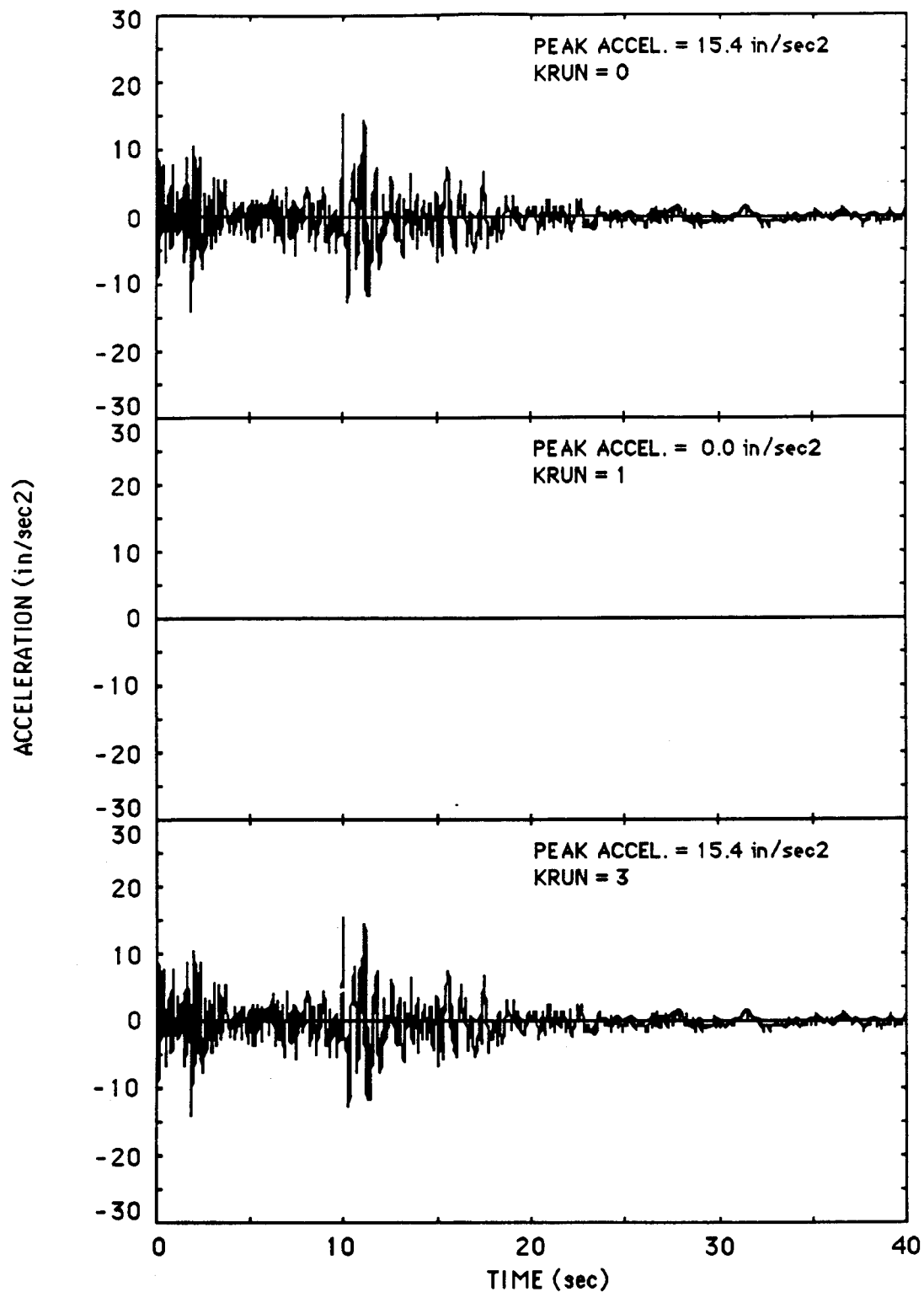


Figure 6.D-33
Treasure Island soil column: vertical motion for compacted soil model from time domain calculation

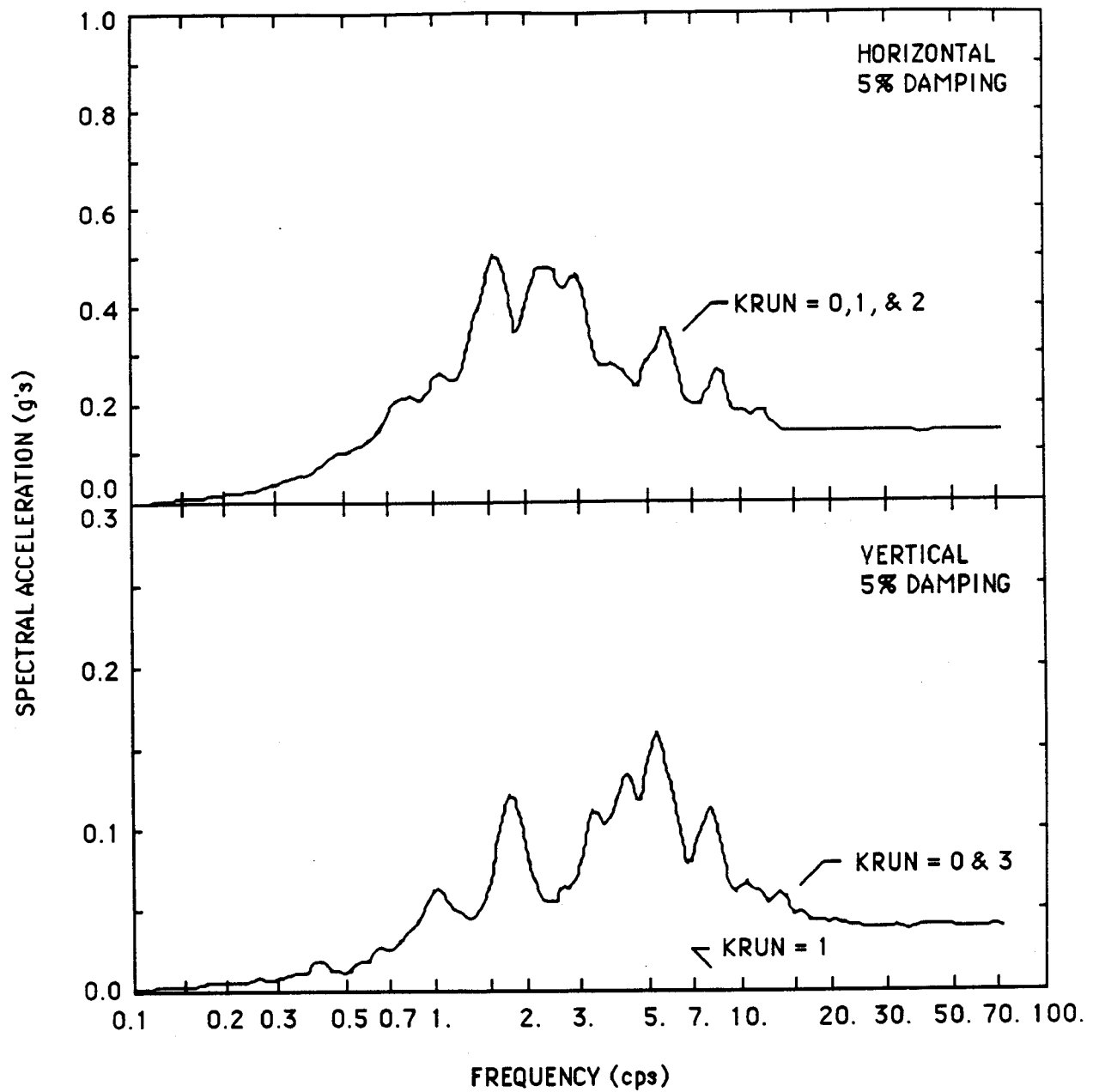


Figure 6.D-34
 Treasure Island soil column: response spectra for compacted soil model from time domain calculation

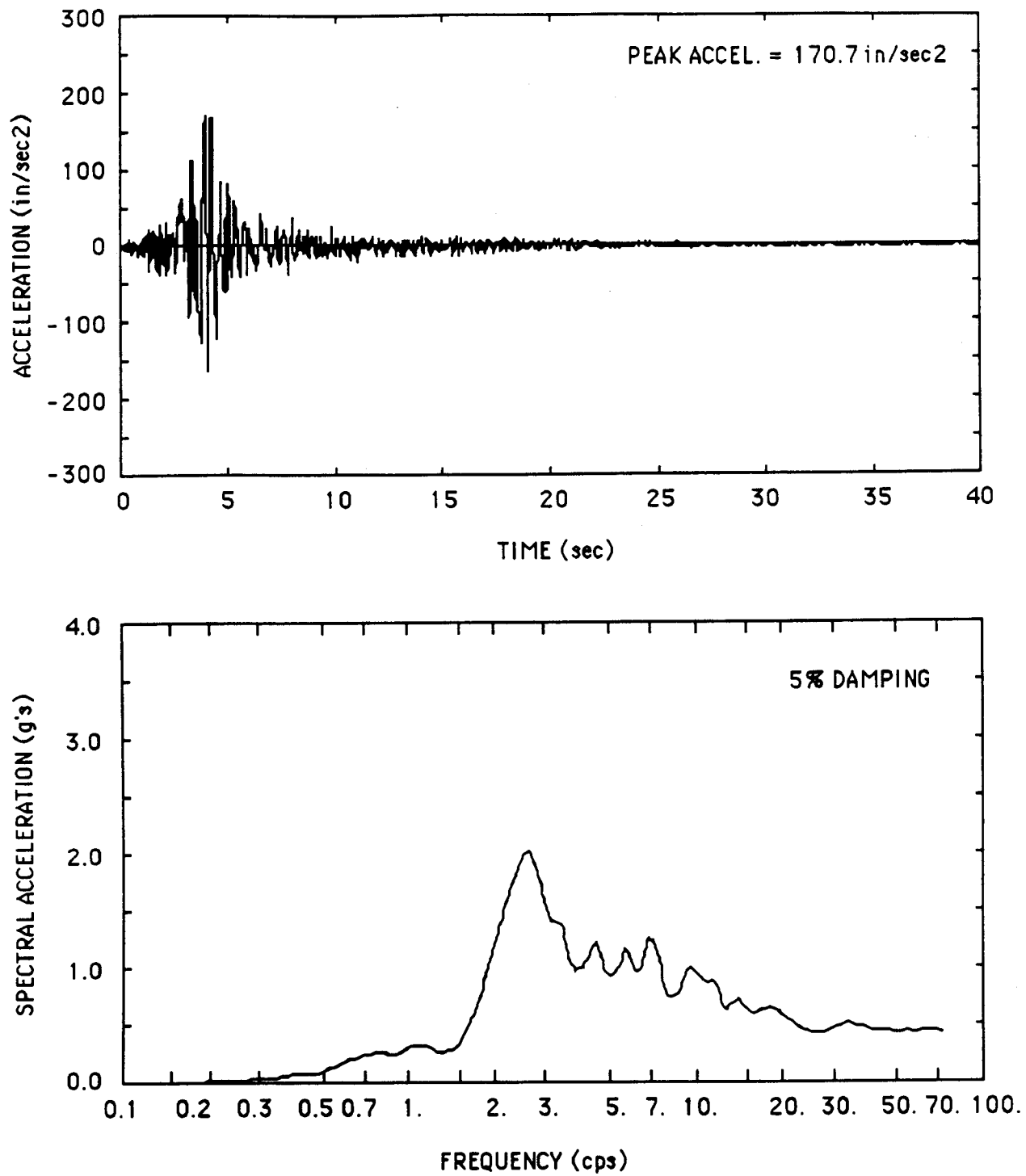


Figure 6.D-35
Gilroy 1, input motion: measured horizontal motion (channel 1)

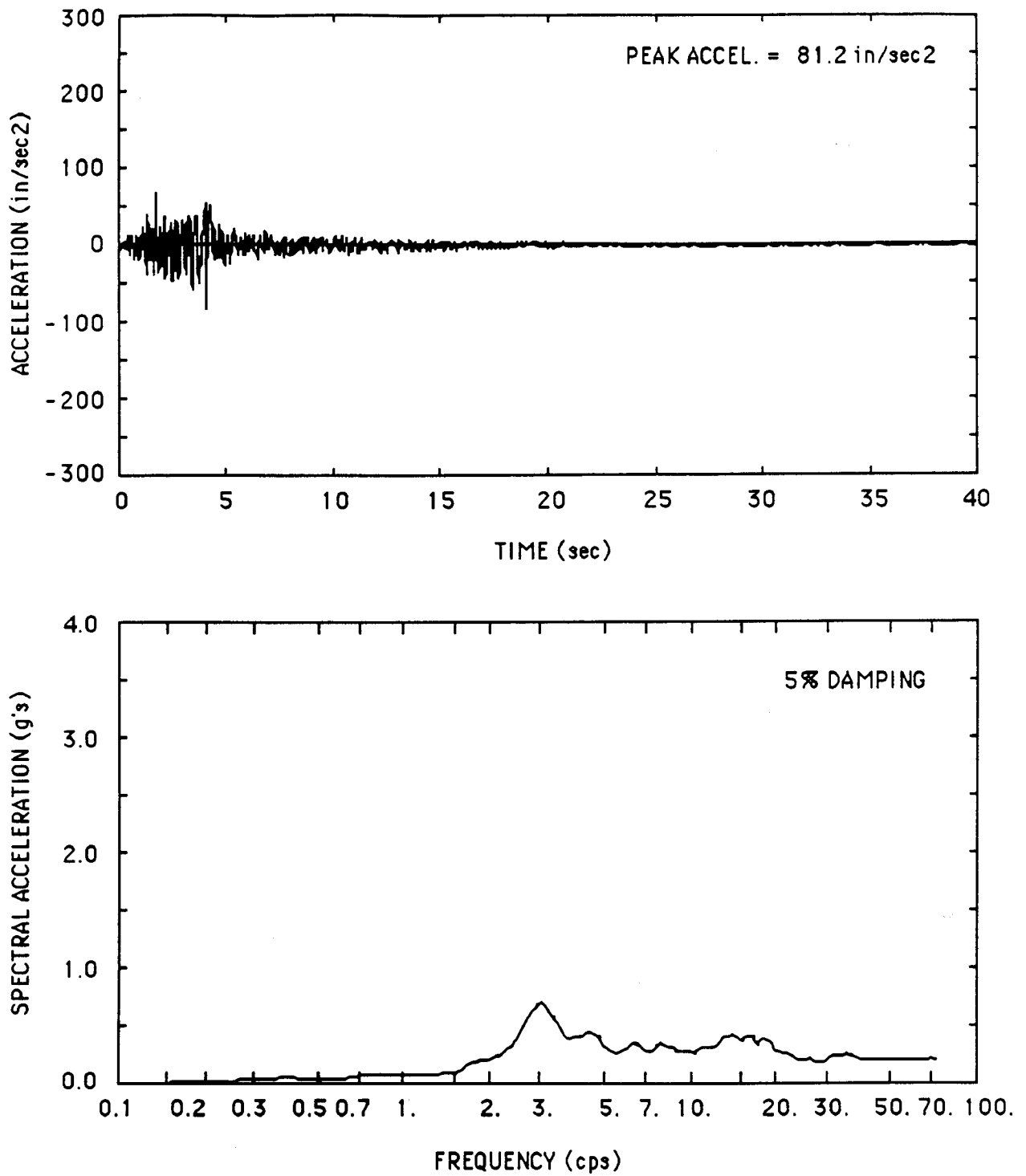


Figure 6.D-36
Gilroy 1, input motion: measured vertical motion (channel 2)

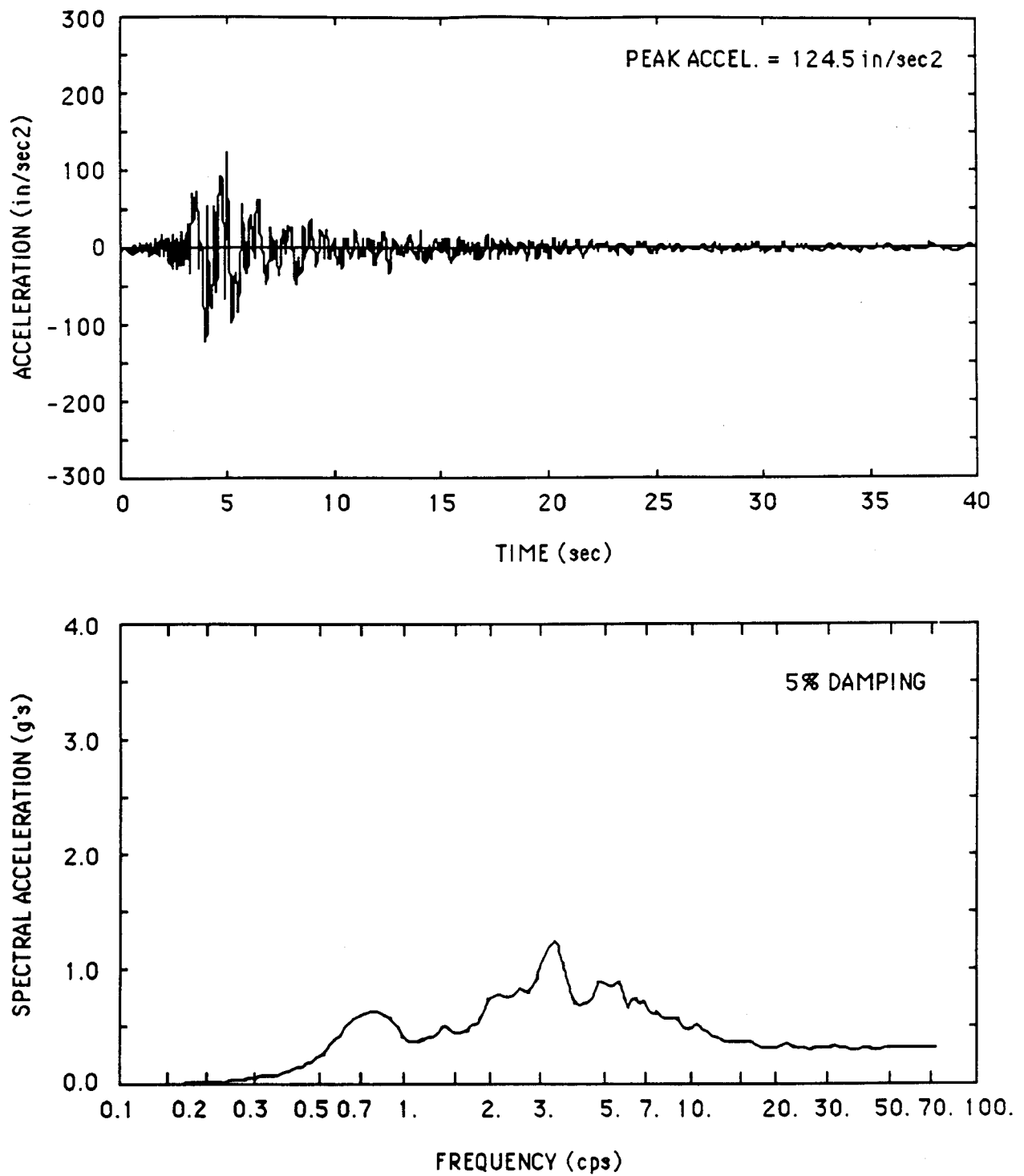


Figure 6.D-37
Gilroy 2 soil column surface motion: measured horizontal motion (channel 1)

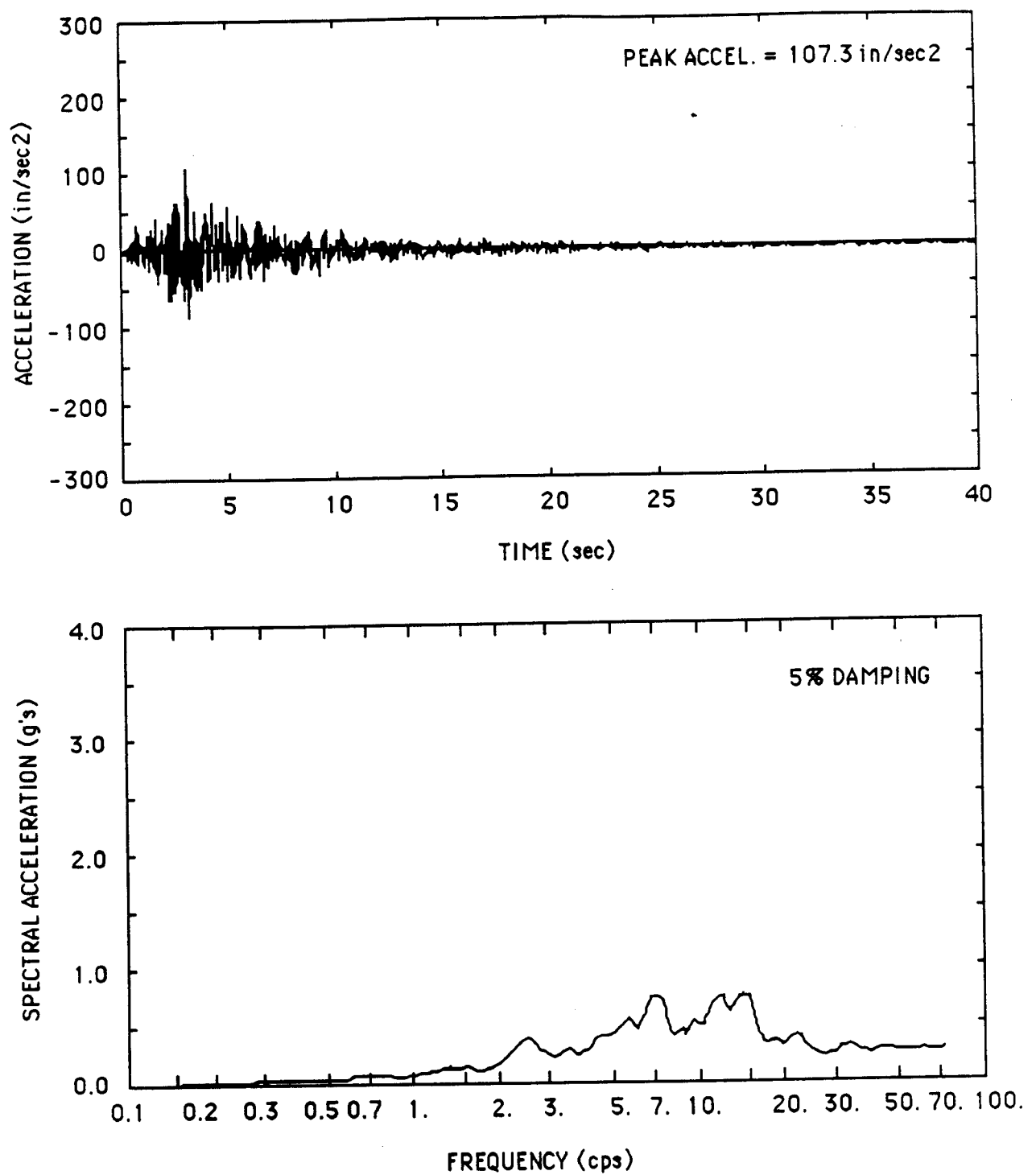


Figure 6.D-38
Gilroy 2 soil column surface motion: measured vertical motion (channel 2)

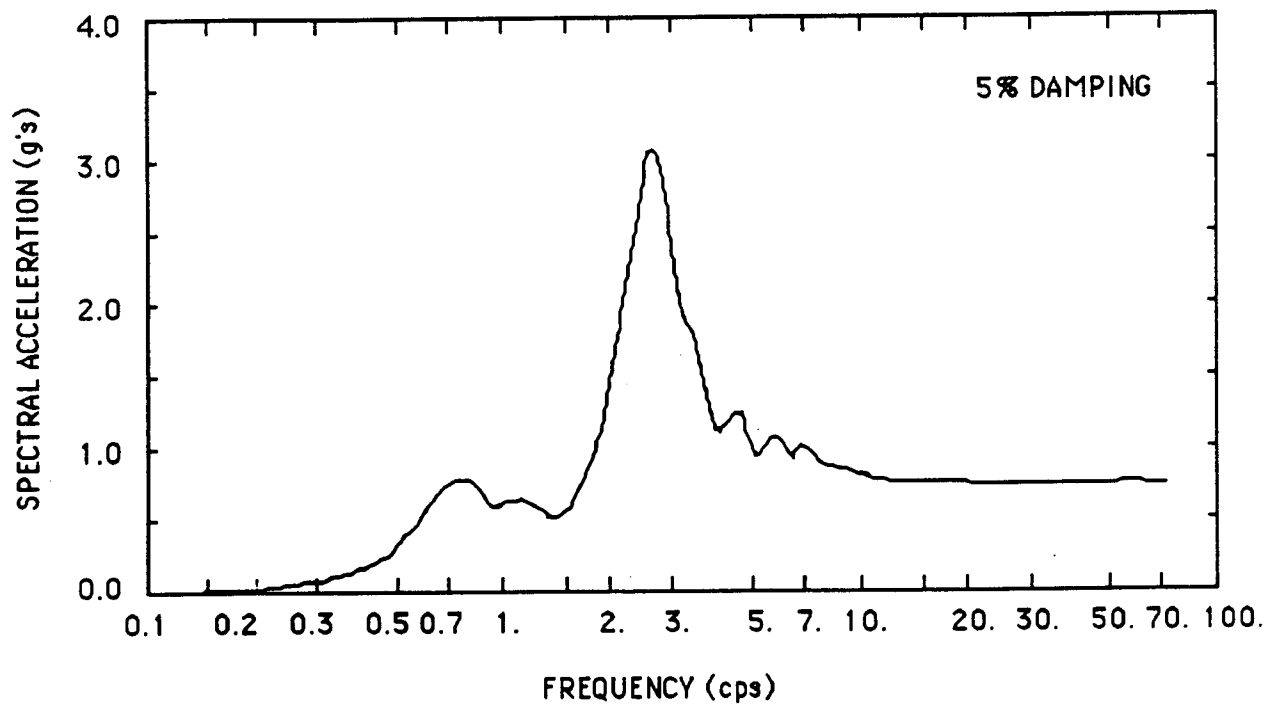
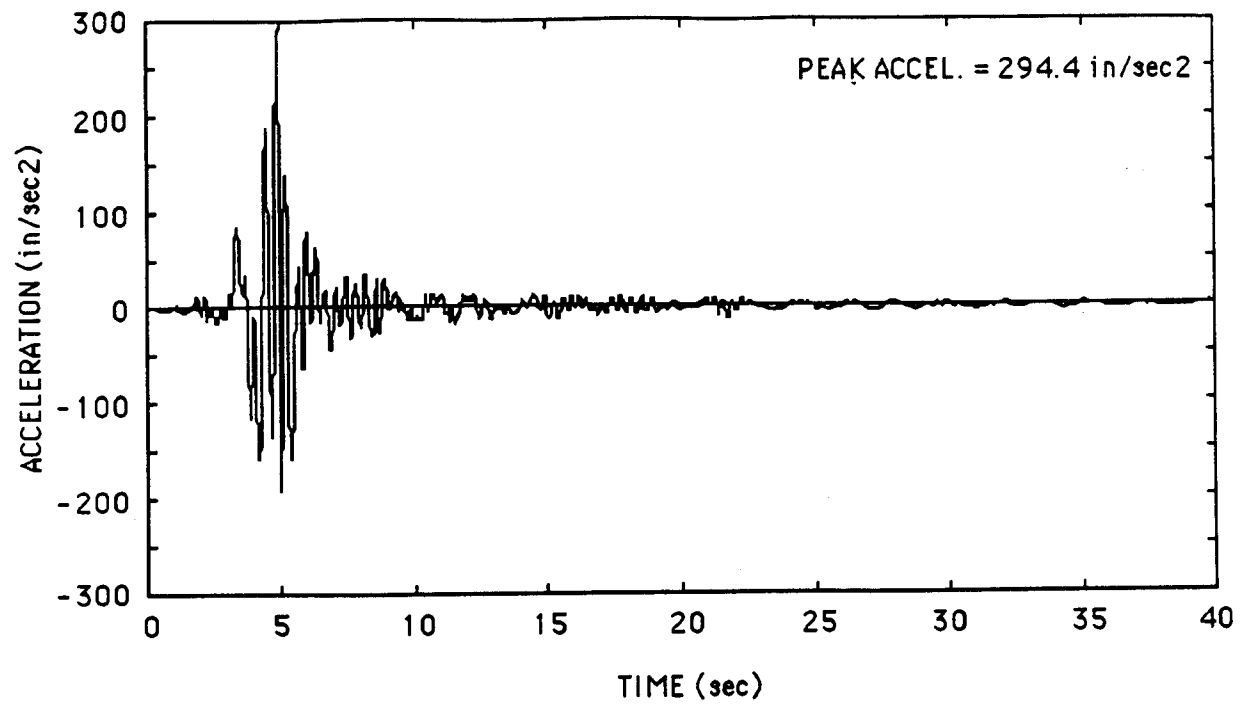


Figure 6.D-39

Gilroy 2 soil column: horizontal surface motion using Gilroy 1 channel 1 input with iterated linear convolution and Stokoe soil degradation properties

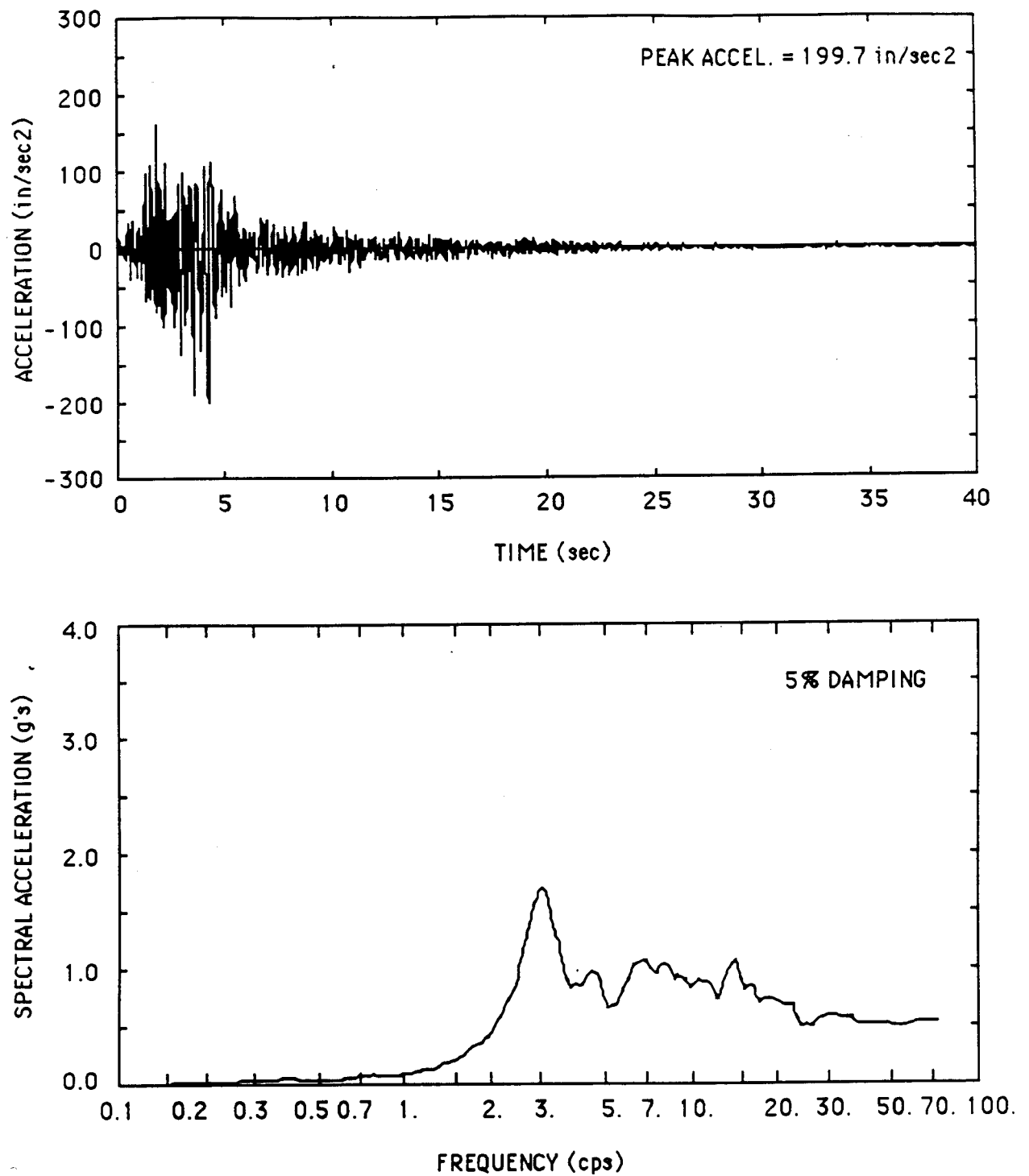


Figure 6.D-40
Gilroy 2 soil column: vertical motion using Gilroy 1 channel 2 input with linear convolution,
no soil degradation and 2% hysteretic damping

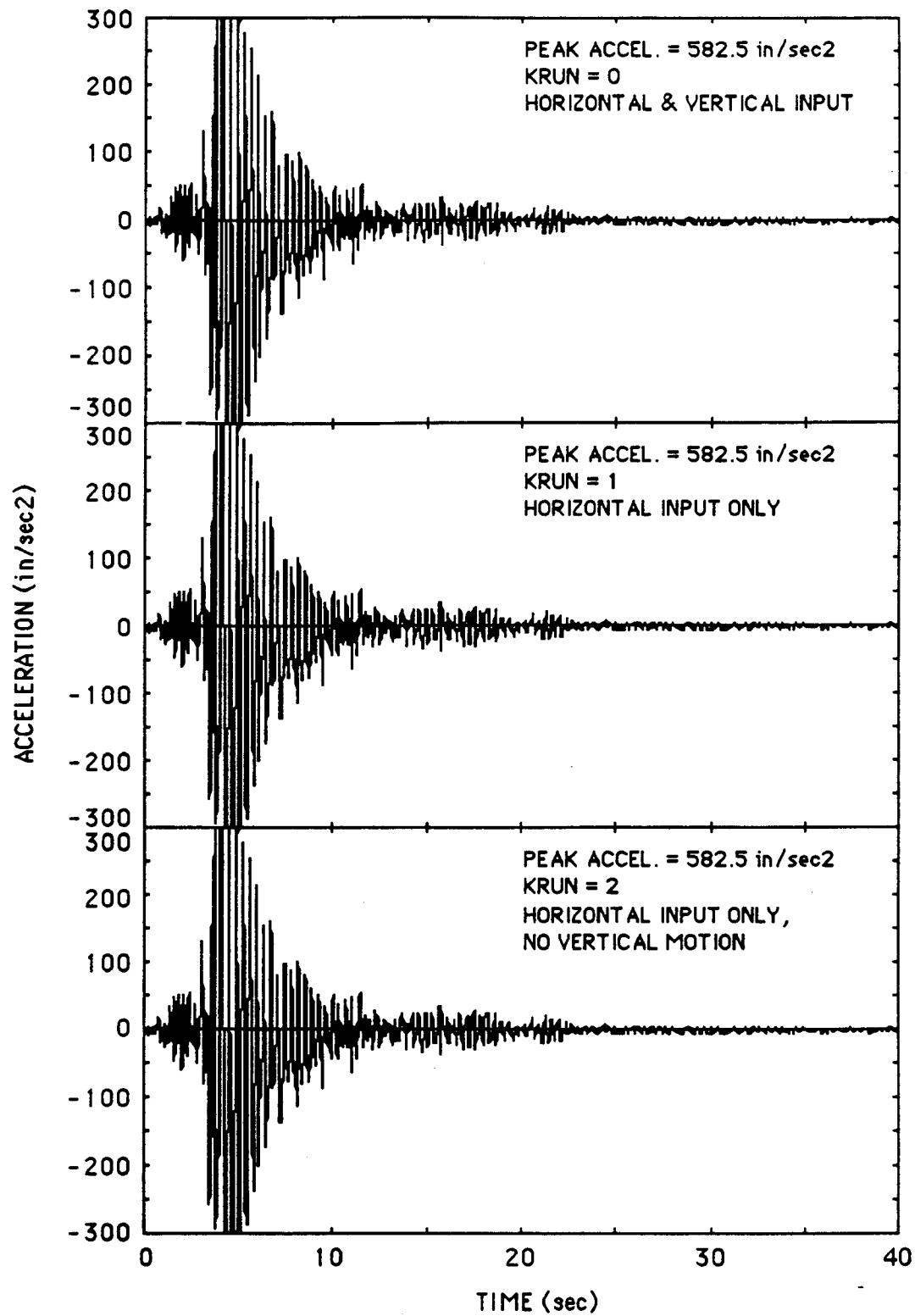


Figure 6.D-41
Gilroy 2 soil column: horizontal motion for elastic soil column from time domain calculation

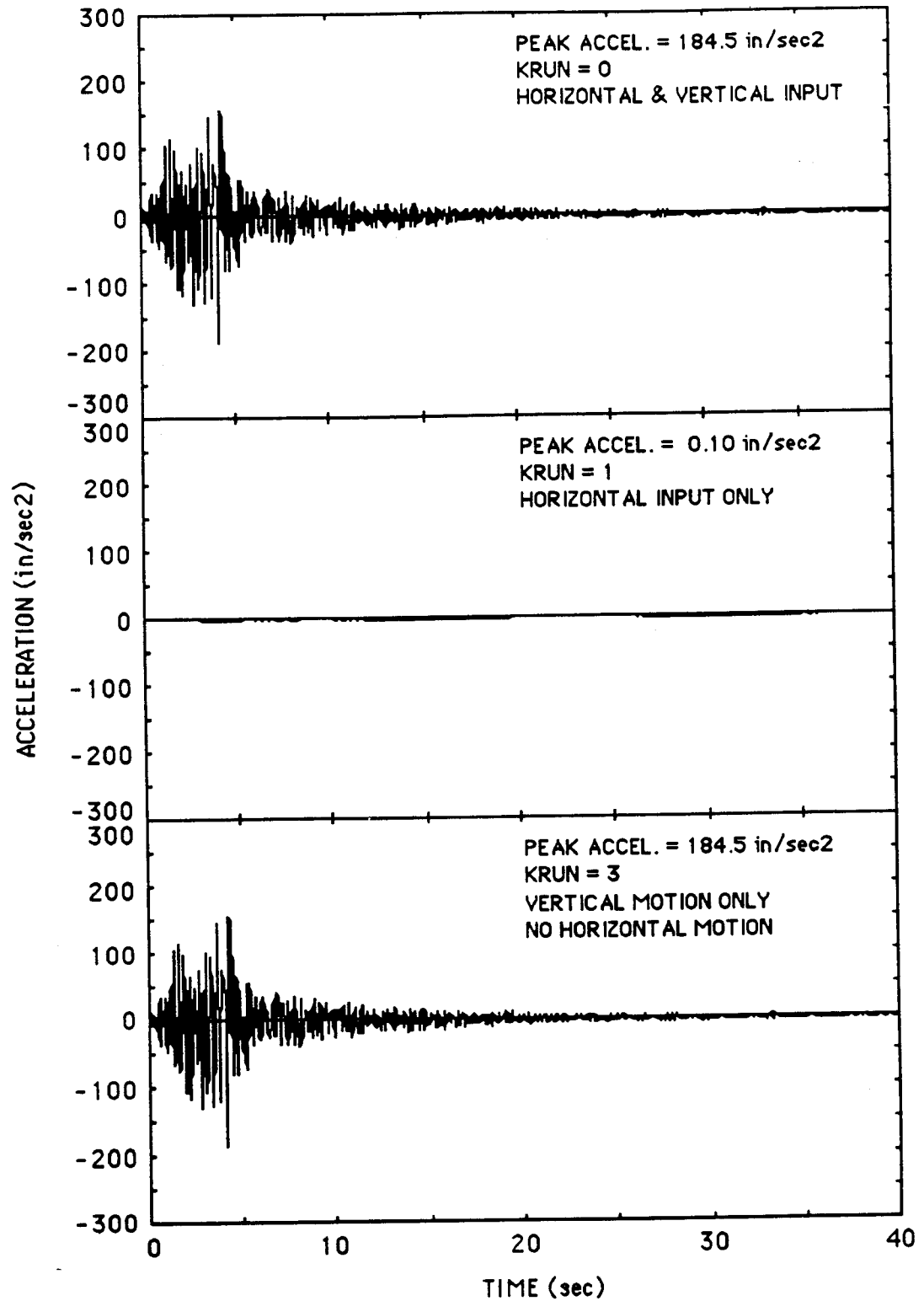


Figure 6.D-42

Gilroy 2 soil column: vertical motion for elastic soil column from time domain calculation

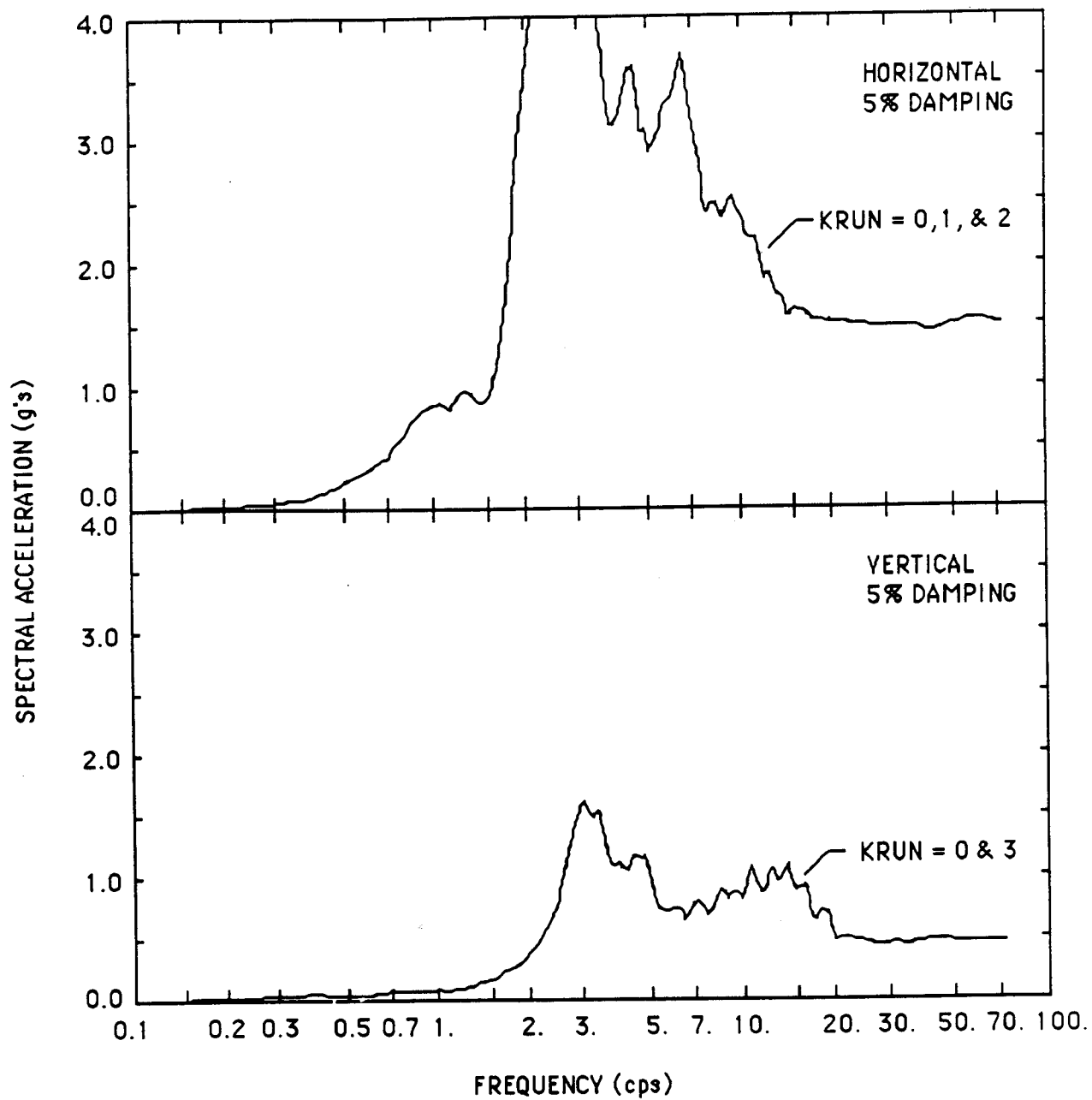


Figure 6.D-43

Gilroy soil column: spectra for elastic soil model from time domain calculation

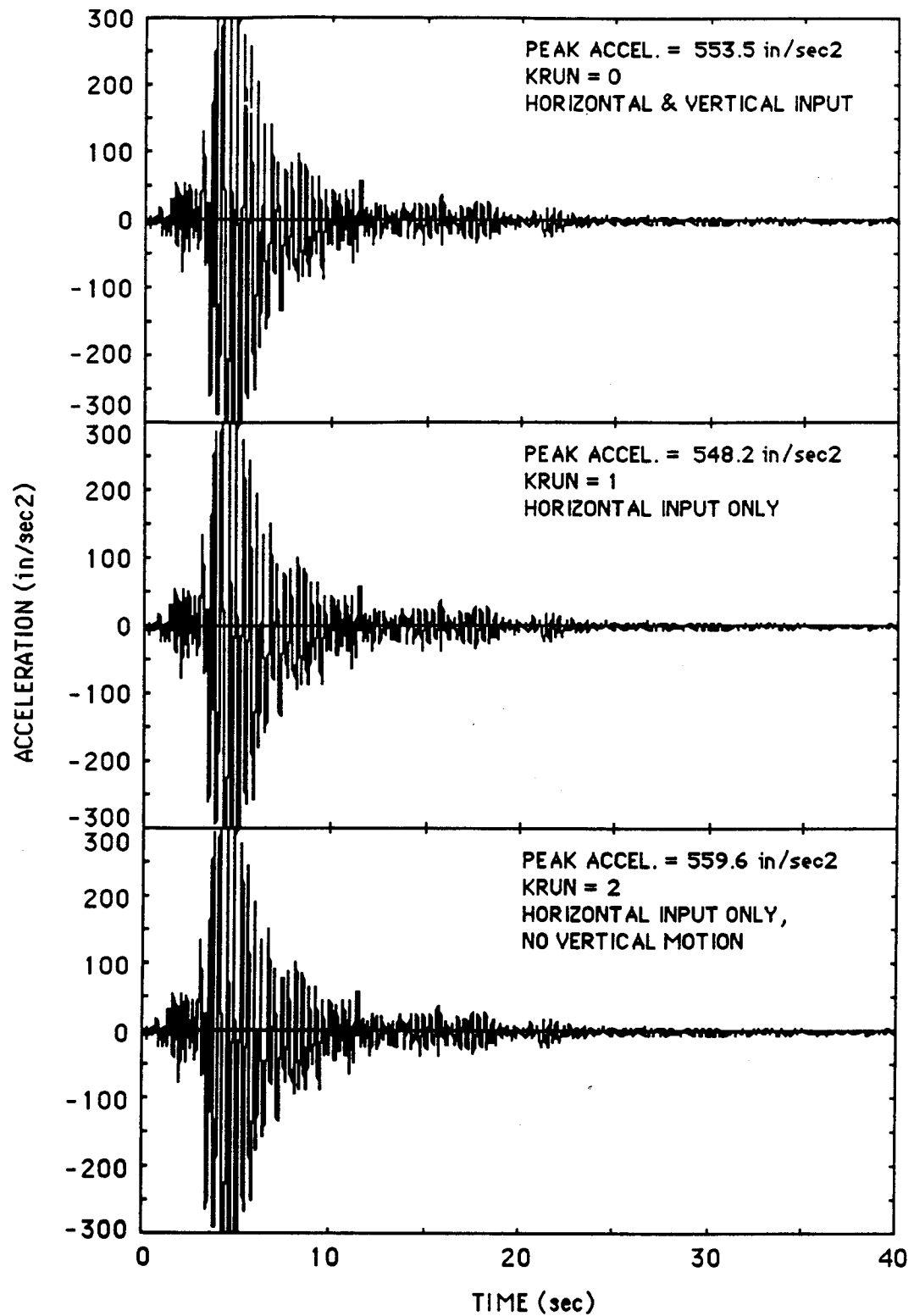


Figure 6.D-44

Gilroy 2 soil column: horizontal motion for Coulomb-Mohr soil column from time domain calculation, non-associated flow rule

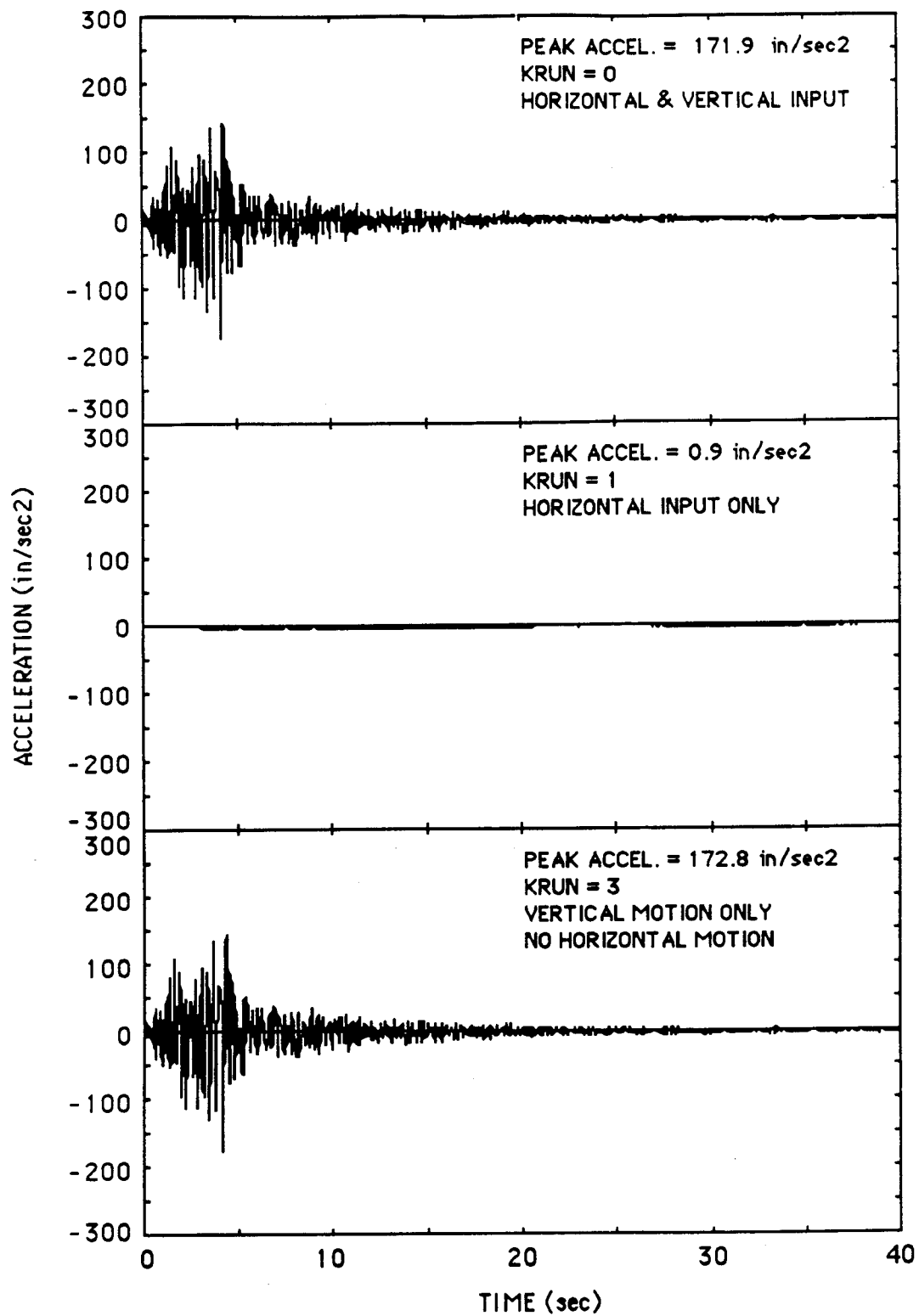


Figure 6.D-45

Gilroy 2 soil column: vertical motion for Coulomb-Mohr soil column from time domain calculation, non-associated flow rule

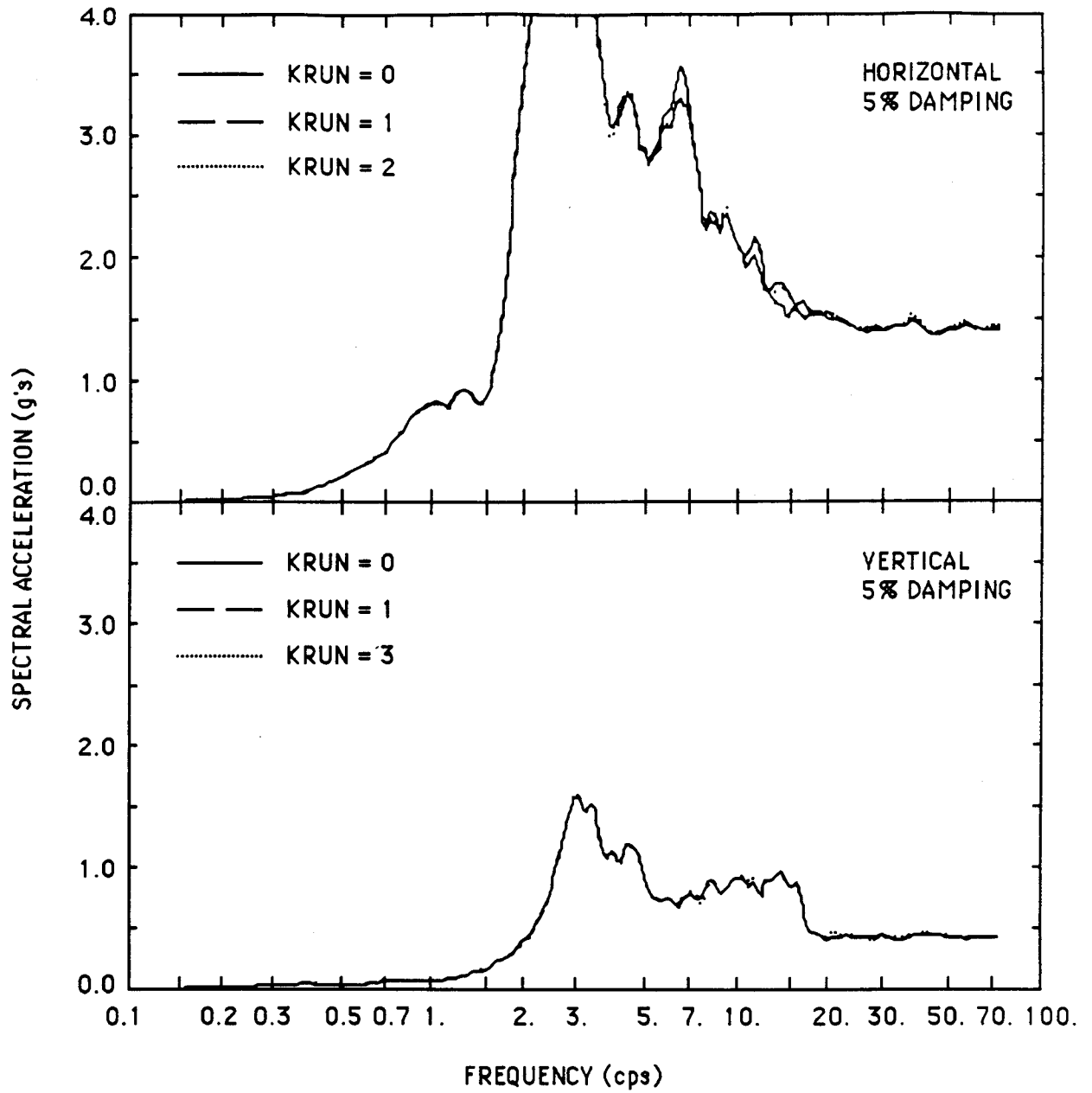


Figure 6.D-46
Gilroy soil column: surface response spectra for Coulomb-Mohr soil model from time domain calculation, non-associated flow rule

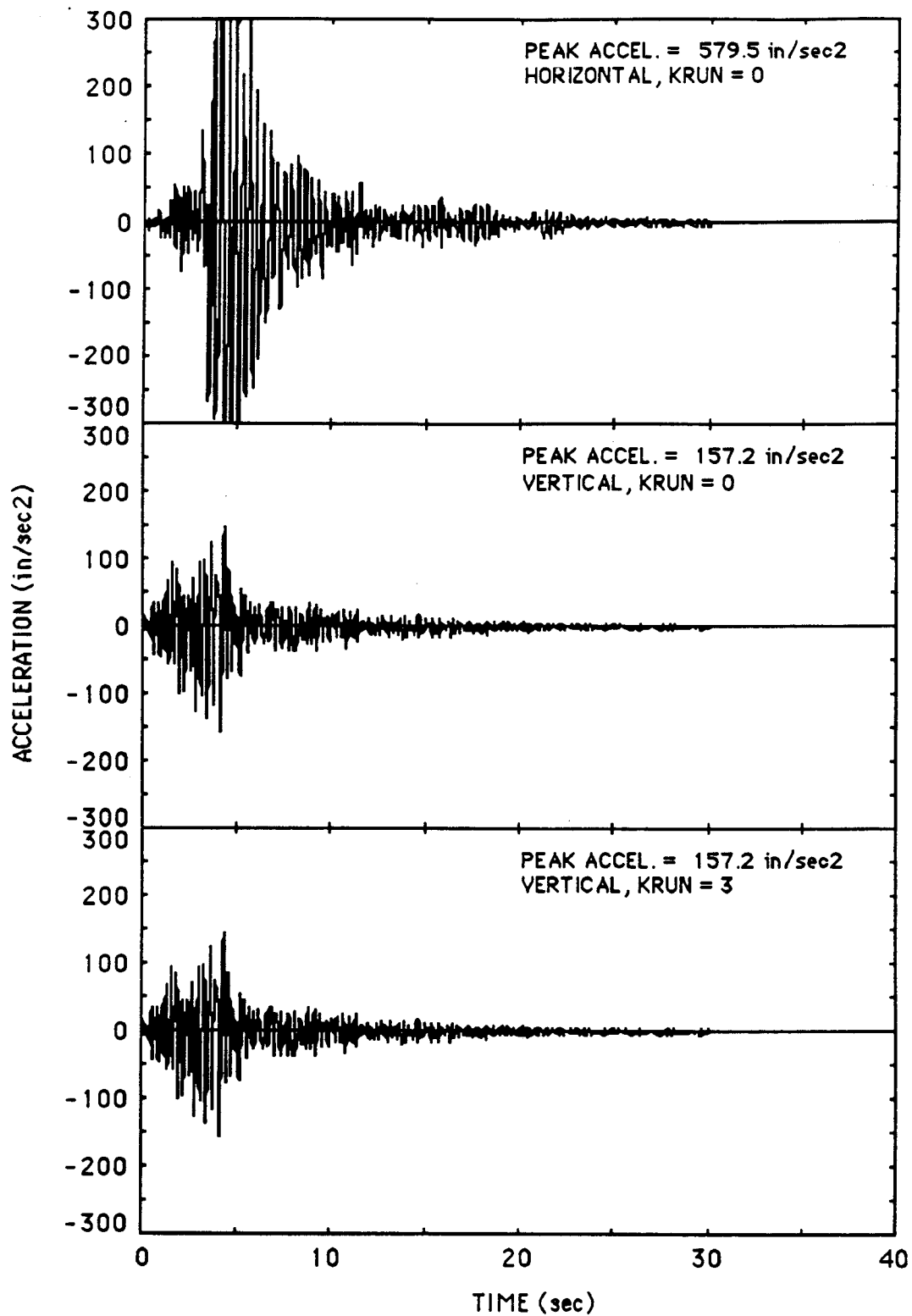


Figure 6.D-47
Gilroy 2 soil column: surface motion for compact soil model from time domain calculation

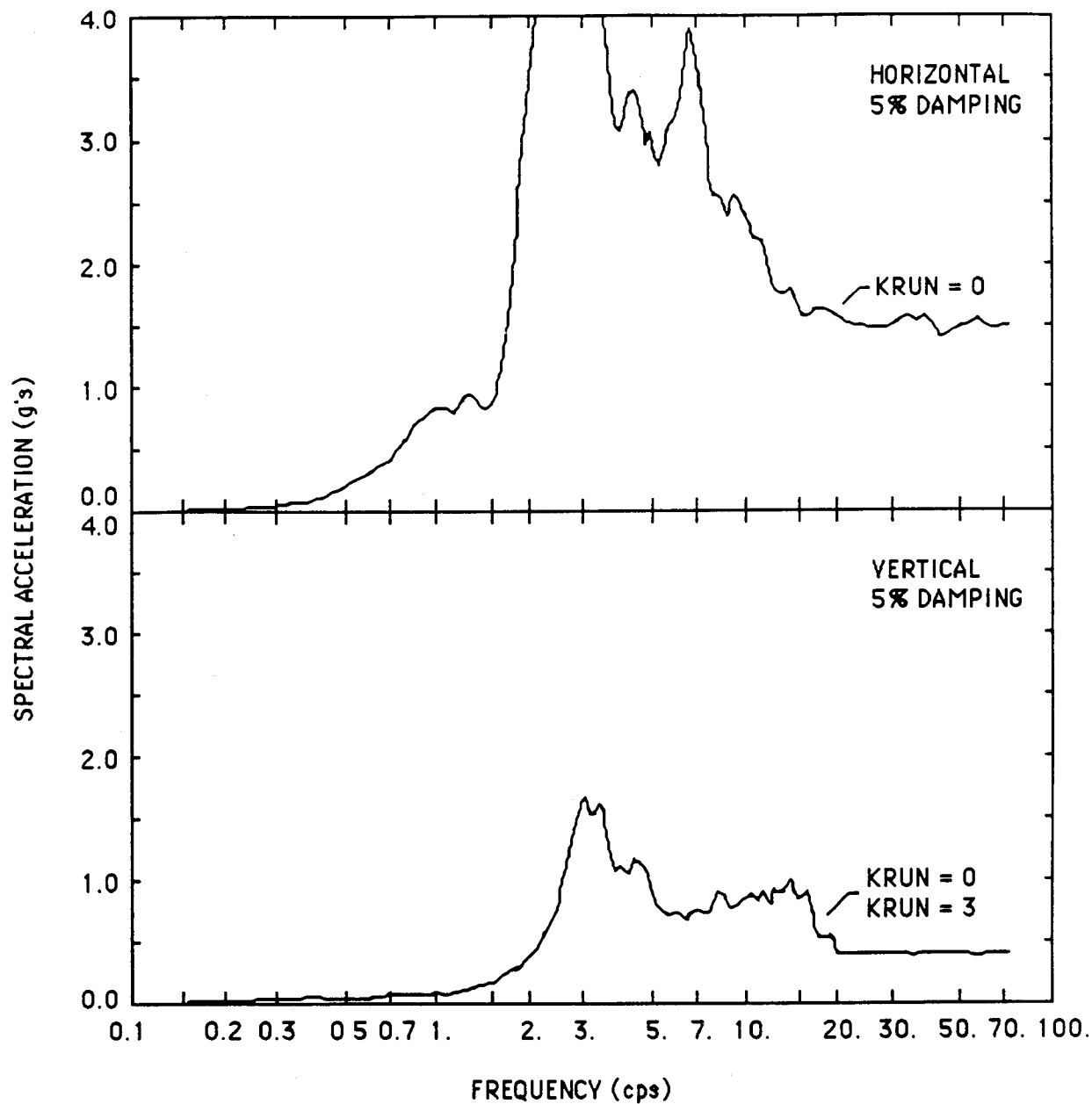


Figure 6.D-48

Gilroy soil column: spectra for compact soil model from time domain calculation

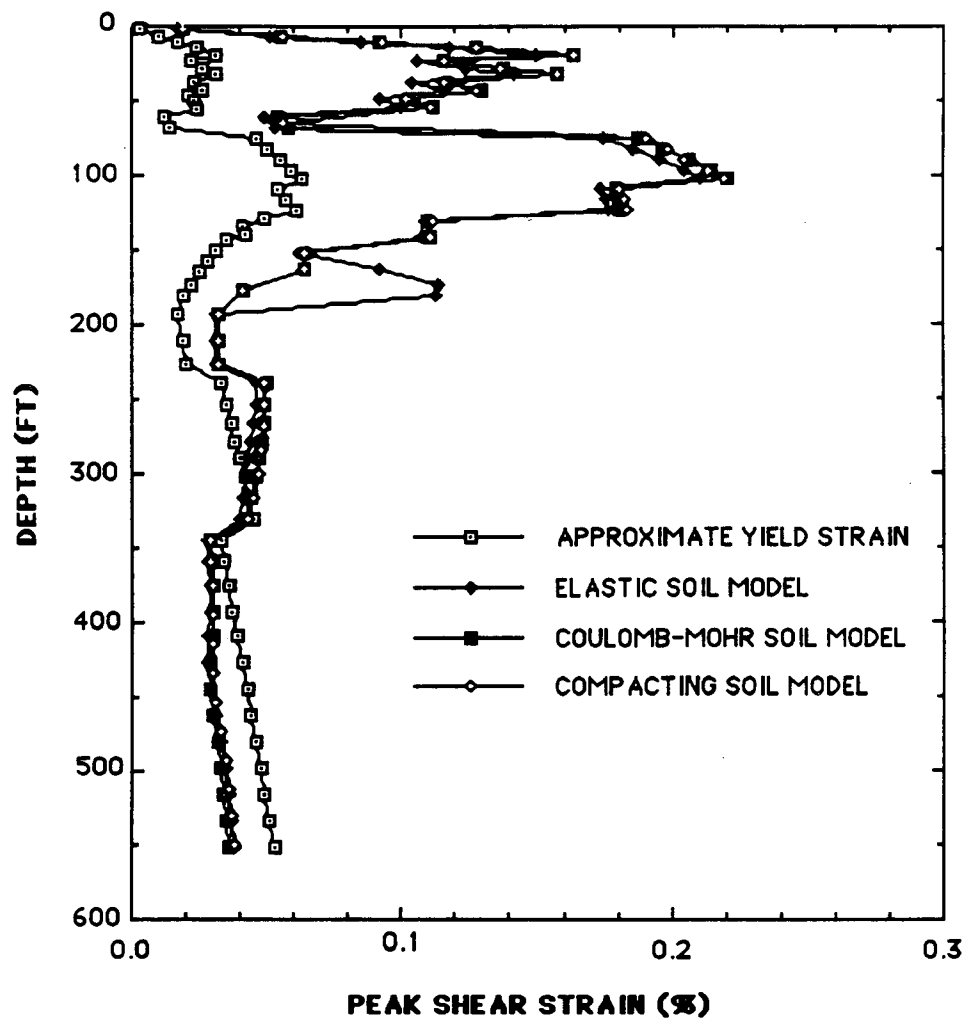


Figure 6.D-49
Peak shear strains for Gilroy soil column

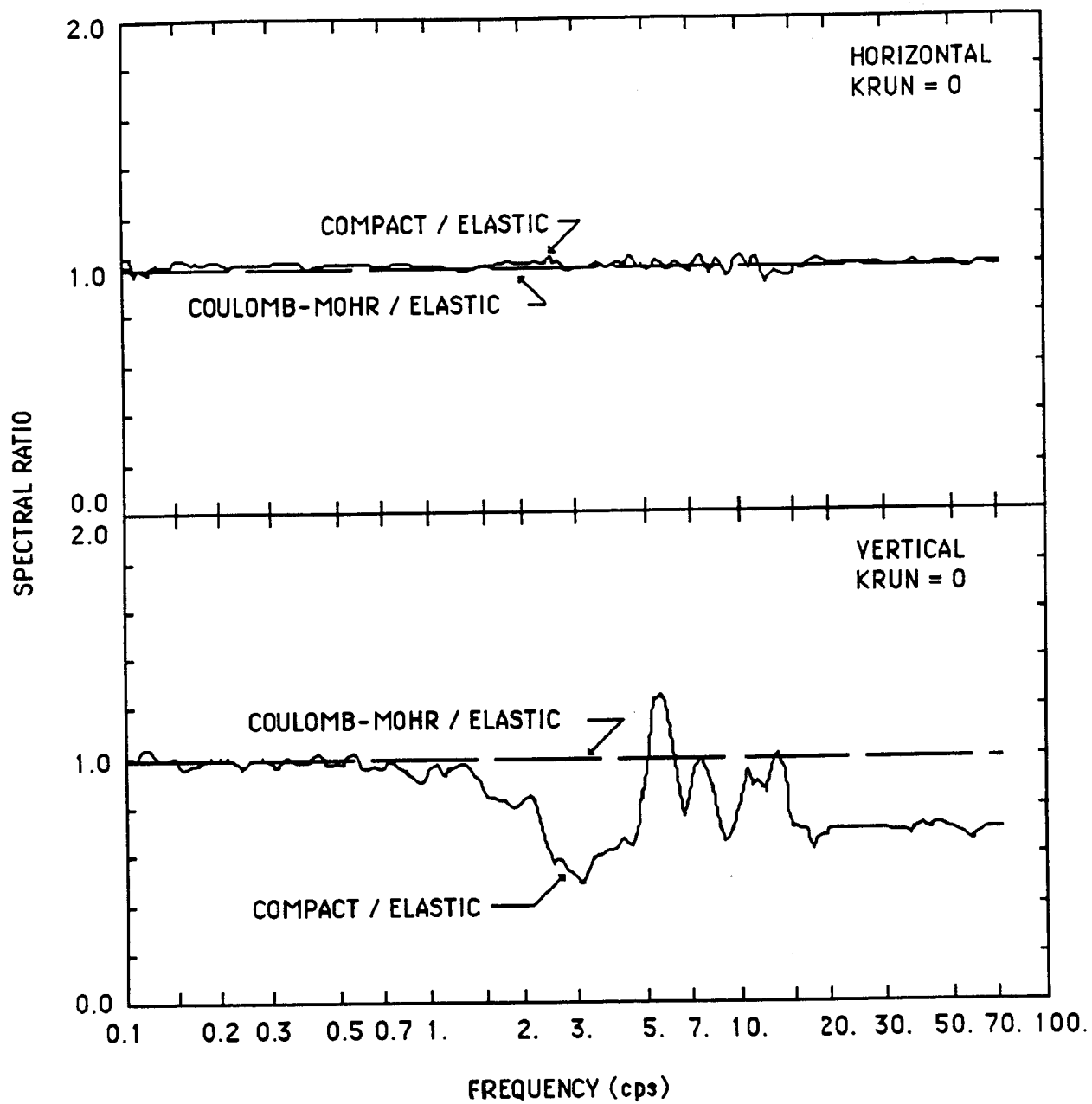


Figure 6.D-50
Treasure Island spectral ratios

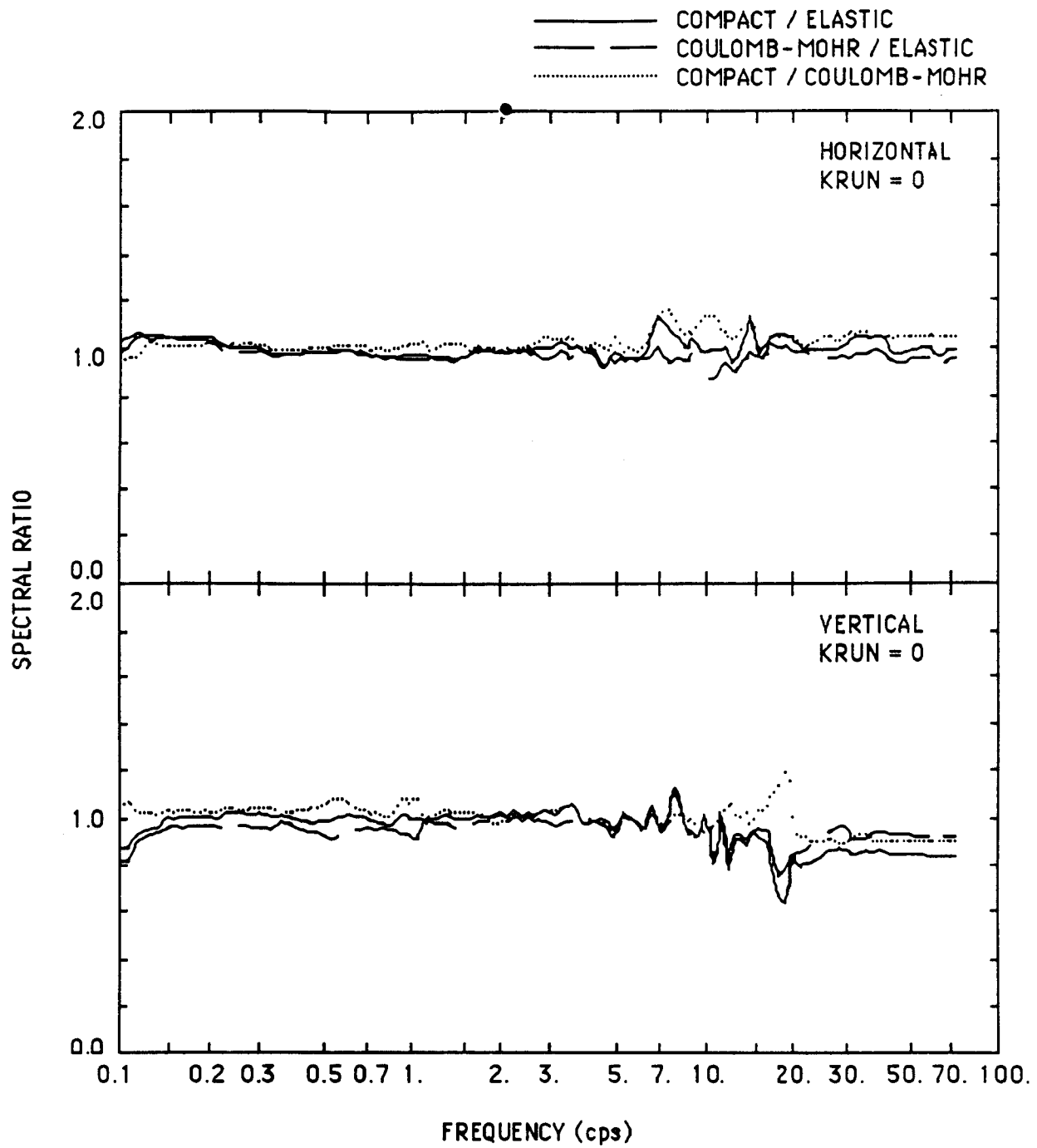


Figure 6.D-51
Gilroy spectral ratios

APPENDIX 7.A

MODELING OF DYNAMIC SOIL PROPERTIES

7.A.1 Introduction

7.A.1.1 Scope

This appendix has the following purposes:

- It provides a reference document for other workers engaged in either generic or site-specific studies drawing on both previously published data and new data developed as part of this project;
- It provides the background necessary for interpreting the field and laboratory data for the three reference sites reported in Appendices 8.A and 8.B for use in the analyses reported in Appendix 6.A;
- It documents the development of the generic modulus reduction and damping curves used in Appendix 6.B;
- It extends a previously developed simple nonlinear soil model to include improved modeling of damping at both low and high strains.

7.A.1.2 Definition of Terms

Ideally the relationships between stress and strain that are used in site response analyses would form part of a single integrated model of soil behavior which would accommodate complex three-dimensional cyclic loadings as well as simple monotonic loadings to failure. However, while some progress has been made towards this goal, as a practical matter, simpler relationships which describe only the subset of behavior most applicable to a particular application are commonly used in practice. Thus, in analyses of site response which have generally assumed that motion can be satisfactorily represented by vertically propagating shear waves, there is only a need for appropriate relationships between shear stress and shear strain in horizontal layers. Further, motion is only considered to occur in a single vertical plane so that the possible effects of a second, orthogonal component of motion is neglected. If any consideration is given to vertical motions, these are also decoupled. Some comments on the effects of multi-directional shaking and the selection of properties for use in analysis of vertical motions are included at the end of this appendix but otherwise we consider only simple shear stress-shear strain relationships.

While nonlinear analyses may require only a single relationship that relates shear stress and shear strain under irregular cyclic loadings with appropriate generation of hysteretic damping, construction of such relationships or "models" is non-trivial and, as a practical matter, such models are usually based on the results of laboratory tests conducted using uniform, symmetrical loadings. Such tests also provide the basis for the description of the properties used in "equivalent linear" analyses (Seed and Idriss, 1969). The first two and a quarter cycles of such a test are depicted in Figure 7.A-1. In this and subsequent figures the normalized shear stress is shown on the y axis. This is the shear stress normalized by the applicable maximum shear stress, $\tau_{\max}$. The equivalent linear shear modulus is defined as the secant shear modulus after a given number of cycles with a constant value of the cyclic shear strain. In Figure 7.A-1, the modulus is shown after two cycles with a cyclic shear strain of 0.1 percent. The corresponding equivalent hysteretic damping ratio is given by the following expression:

$$D = \frac{1}{4\pi} \frac{\text{area within loop}}{\text{area of triangle}}$$

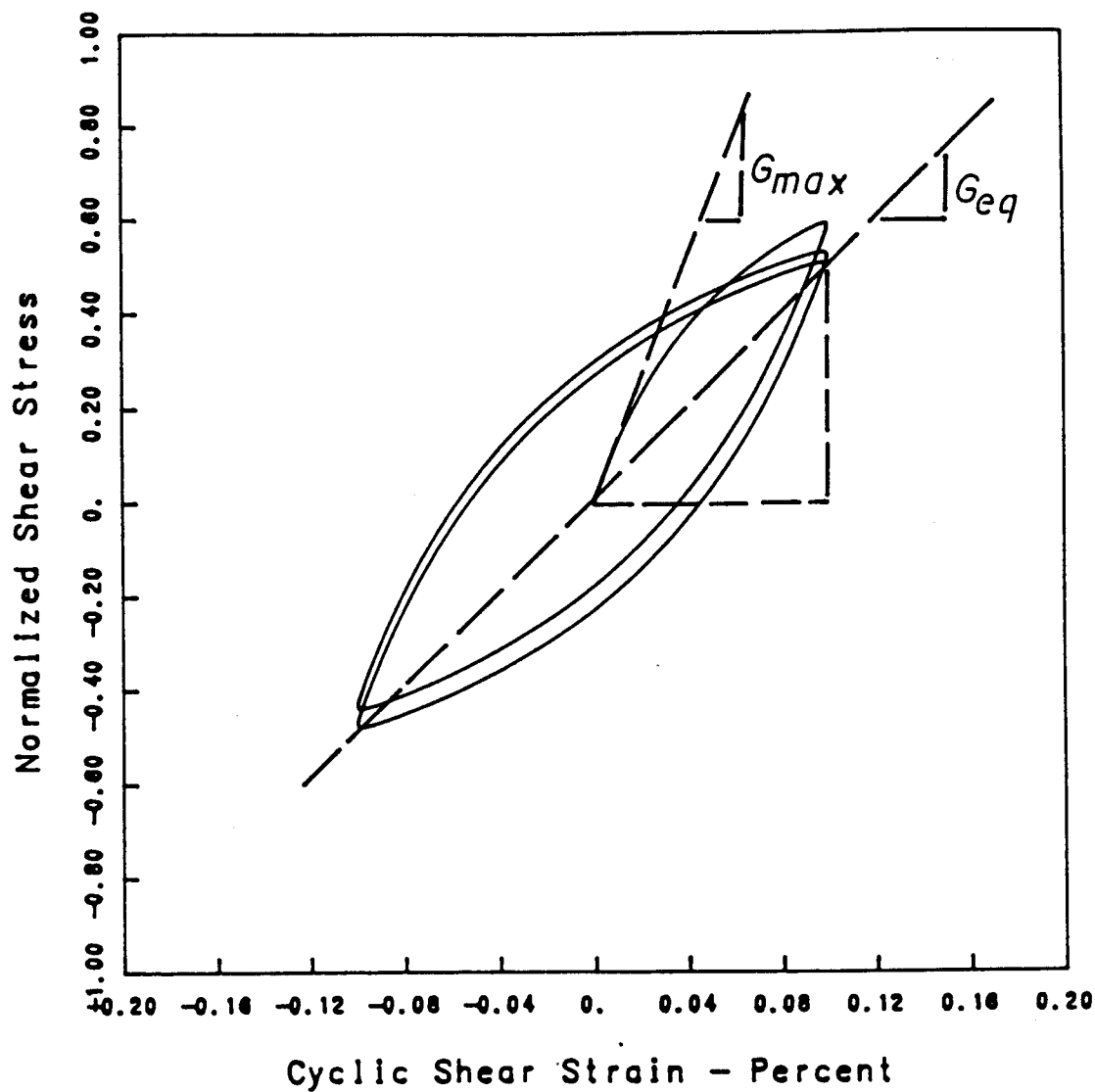


Figure 7.A-1

Typical Shear Stress-Shear Strain Relationship

When plotted at the same scale as shown in Figure 7.A-1, the stress-strain loops at very small strains collapse to a straight line with a slope equal to the maximum shear modulus, G_{max} , which closely approximates the initial tangent modulus on first loading or on reversal under cyclic loading. As the amplitude of the cyclic shear strain increases, the equivalent linear shear modulus decreases and the damping ratio increases. It is frequently convenient to normalize the shear modulus at a given strain level by dividing it by G_{max} and if the resulting quantity G / G_{max} and the damping ratio are plotted against cyclic shear strain, curves such as those shown in Figure 7.A-2 are obtained. Such curves will subsequently be referred to as modulus reduction and damping curves.

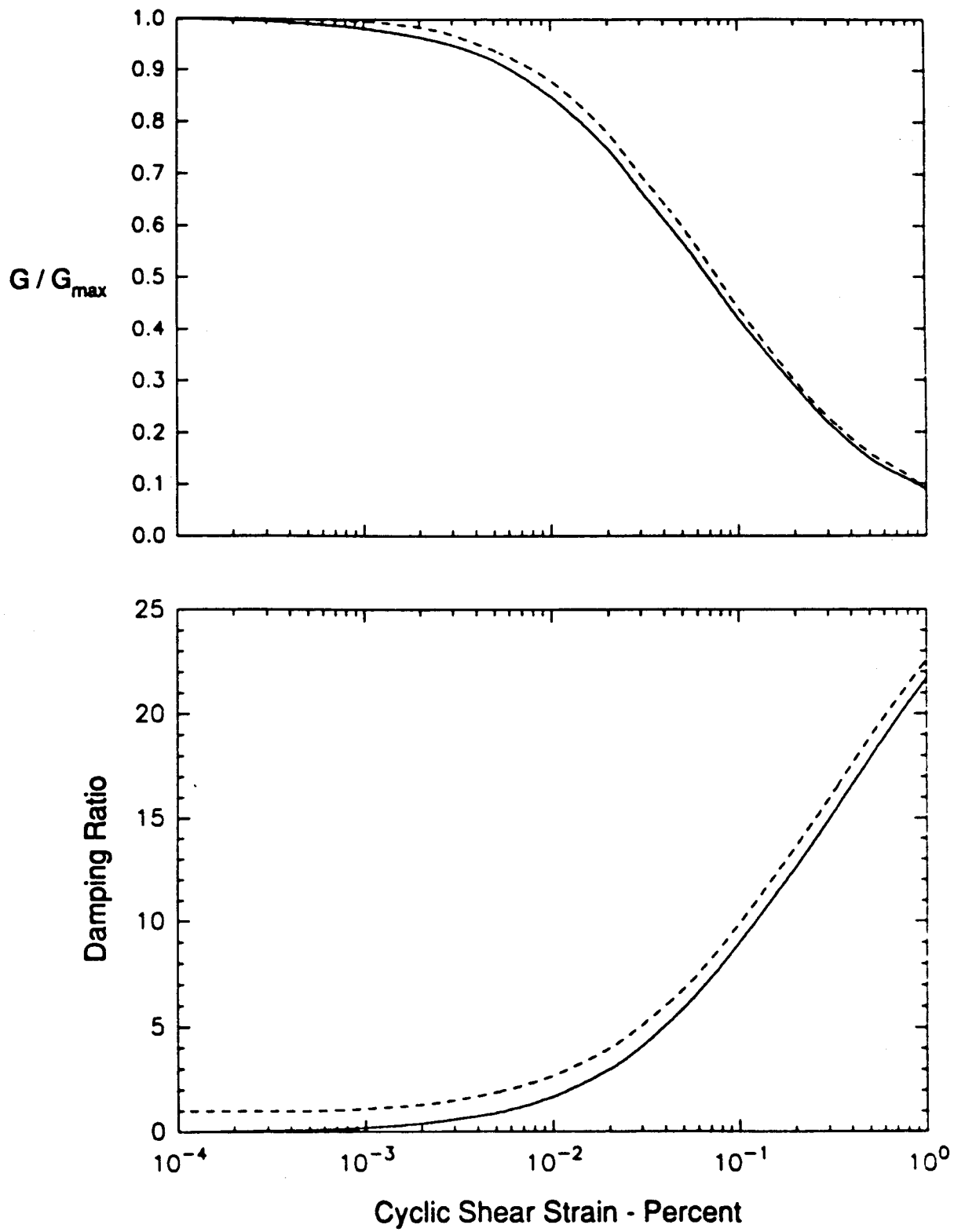


Figure 7.A-2
Typical Modulus Reduction and Damping Curves

Modulus reduction and damping curves of the type shown in Figure 7.A-2 are used directly in equivalent linear analyses in which the modulus reduction factor and the damping ratio are simply “looked up” as a function of the average cyclic shear strain over the duration of excitation for the layer in question. Since the average strains are not known at the outset of the analysis, initial values are assumed and the analysis iterates until “strain compatible” values are found. Conventionally the “average” strain has been taken to be 0.65 times the peak strain in the layer but the question of how to obtain the average strain has recently become more contentious as discussed in Appendix 6.B. In any case, the concept of equivalent linear analyses requires that fixed values of the modulus reduction factor and damping ratio be used throughout the analysis. Thus it is assumed that the modulus reduction factors and damping ratios are independent of the frequency and the number of cycles of loading although the values used are supposed to be obtained using representative loading rates and load histories. These are not unreasonable assumptions in the context of an analytical procedure in which other broader assumptions are made but the notion that the secant shear modulus and the damping ratio are actually independent of the frequency or rate of loading and the number of cycles of loading is not entirely correct.

The assumption that damping in soils is purely hysteretic in nature and is independent of the frequency of loading is not inconsistent with the results of many laboratory studies but gives rise to an interesting inconsistency in commonly used modulus reduction and damping curves. While equivalent linear analyses allow the use of independently derived and possibly inconsistent modulus reduction and damping curves, these curves cannot be completely independent of each other if damping is purely hysteretic. In particular, at strains less than what is sometimes called the elastic threshold where the modulus reduction factor remains equal to one, soils, in some sense at least, exhibit linear behavior and the damping ratios at these strains might be expected to be zero, as shown by the full lines in Figure 7.A-2. Actual measurements, however, have always shown small but finite values of damping in this strain range, as shown by the broken lines in Figure 7.A-2. A possible explanation for this behavior and further discussion of the effects of the frequency and number of cycles of loading is offered later in this appendix.

While the equivalent linear or secant values of the shear modulus that are derived from an appropriate value of $G_{\max}$ and an appropriate modulus reduction curve are in some sense artificial values that make sense only in the context of equivalent linear analyses, it turns out that these values can also be used to define the “backbone” curve used to construct a nonlinear soil model. While, strictly speaking, the backbone curve should be an extension of the shear stress-shear strain relationship for the first quarter cycle of loading as shown in Figure 7.A-1, this curve should be obtained at the correct rate of loading and may be modified by subsequent cyclic loading. Thus, the backbone curve appropriate for use in a model of behavior under rapid cyclic loading and the locus of the end points of stress-strain loops such as those shown in Figure 7.A-1 are not necessarily the same as the shear stress-shear strain curve obtained for a monotonic loading to failure at any fixed rate of loading. It might be noted that in conventional soil mechanics, tests in which specimens are loaded to failure under monotonic loadings are usually called “static” tests, regardless of the rate of loading. Similarly, tests conducted to simulate earthquake loadings are usually referred to as “dynamic” tests although the fact that they are conducted using cyclic loading may be equally or more important than the rate of loading. Thus, the properties required for either equivalent linear or nonlinear analyses are usually referred to as “dynamic properties.”

It should also be noted that the concept of equivalent linear analyses does not require the use of a shear strength and that it is possible to iterate to strain-compatible shear moduli and damping ratios which yield computed values of the shear stress which exceed conventional estimates of the shear strengths in some layers. This is not necessarily inconsistent, since conventional estimates of the “static” shear strength may err on the low side, in part because rate of loading effects are neglected, but also because it is normally conservative to err on the low side in estimating shear strengths. It is not, however, conservative to reduce the shear modulus in order to avoid exceeding a low estimate of the shear strength in an equivalent linear analysis of site response or to use a low estimate of the shear strength as the asymptote for the shear stress-shear strain relationship in a nonlinear analysis.

In summary, then, equivalent linear analyses require estimates of $G_{\max}$ and modulus reduction and damping curves for each layer used in the analysis and this is also the minimum information needed to construct a nonlinear model. Additional information may be required for nonlinear models which follow rate of loading and cyclic loading effects in greater detail. This information is usually rather specific to the particular nonlinear model that is used but two relatively simple examples are discussed in Section 7.A.5.

7.A.1.3 Measurement Techniques

For screening and preliminary site specific studies it may be adequate to estimate $G_{\max}$ or the shear wave velocity, V_s , and to use standard modulus reduction and damping curves of the kind that are presented in Section 7.A.6.

The shear modulus at small strains, $G_{\max}$, and the shear wave velocity V_s , are related by the expression:

$$G_{\max} = \rho v_s^2$$

where ρ = the mass density, so that if either $G_{\max}$ or V_s are known or can be estimated, the other can be readily obtained.

Several empirical expressions are available for $G_{\max}$ as discussed in Section 7.A.5 and empirical correlations between V_s and various other soil properties are given, for instance by Sykora and Koester (1988) and Dickenson and Seed (1992). Such correlations should be used with great caution, however, since V_s can be affected very significantly by factors such as age and stress history which are not reflected in simple measures or indexes of soil properties.

For final site-specific studies it is imperative that both $G_{\max}$ and the modulus reduction and damping curves be measured for the actual soils at the site if uncertainties in these quantities are to be minimized. Three general approaches are available: in situ measurements, laboratory measurements, and backcalculation from observed ground motions.

7.A.1.3.1 In Situ Measurements. While there are obvious advantages to making all measurements of properties in situ, the only parameters that can reliably be measured in the field at present (1992) are the shear wave velocity, V_s , and the compression wave velocity, V_p . While very significant advances have been made in measuring low strain damping in situ in recent years, and in this project as reported in Appendix 8.A, standardized measurement and data reduction techniques have not yet emerged. Techniques to measure shear modulus and damping at larger strains in the field have been suggested from time to time but no viable techniques are yet available.

Presently available techniques for measuring velocities in situ include:

SASW—spectral analysis of surface waves—a good approach for use at shallow depths, particularly in soils in which it is difficult to drill and sample.

Crosshole—requires at least two, preferably three or four, closely spaced borings—likely provides the most accurate and repeatable measurements and allows measurement of shear wave anisotropy which may aid in interpretation of lateral stresses—requires accurate measurement of borehole diameter and drift and in practice usually limited to depths of several hundred feet—covered in part by ASTM Standard D4428.

Downhole—can be conducted in a single boring although measurement of borehole diameter and drift is still desirable—can provide very accurate measurements but more susceptible to measurement and interpretation errors than the crosshole technique—problems resulting from deviation of ray path from the direct source-to-receiver path could be minimized by using two or more downhole receivers—effective depth limited by use of surface sources but use of downhole sources could make technique viable to greater depths.

Seismic Cone—similar in principle to downhole—very fast but usually limited to depths of about 100 feet by capacity of thrusting system and rods.

Suspension Logger—desirably run in uncased borehole which limits demonstration of repeatability but results compare favorably with good crosshole and downhole measurements—samples small volumes of soil at high frequencies and shows more local variability than other techniques—can be used at depths up to 1000 feet.

The field investigation of reference sites reported in Appendix 8.A has indicated that generally similar results can be obtained using each of these techniques but that they all have some limitations. Thus it is desirable to use a minimum of two different measurement techniques on critical sites so that they can serve as a cross check on each other and on the occurrence of gross errors.

Most of the above techniques also allow the possibility of measuring low-strain damping in situ and the results obtained in this study and other recent work in this field should be evaluated in order to maximize the possibility of obtaining usable data on damping in conjunction with the measurement of velocities. It should be noted, however, that differing estimates of damping may be obtained from downhole and crosshole tests because of scattering, reflection and transmission effects, and that the effect of the frequency of excitation may need to be taken into account in comparing different field methods as well as in comparing field and laboratory measurements.

7.A.1.3.2 Laboratory Measurements. A variety of laboratory tests are available for measuring both $G_{\max}$ and the modulus reduction and damping curves at larger strains. In earlier work the strain ranges over which most test equipment could operate reliably was limited and different tests would be used to measure $G_{\max}$ and to obtain modulus reduction and damping curves. This has lead on some occasions to obvious inconsistencies but better quality work usually shows relatively good agreement between the results of different types of laboratory tests and newer equipment is generally capable of operating over wider strain ranges.

The most common laboratory tests are:

Resonant Column—Usually applied in torsion on solid cylinders but can be used for longitudinal excitation also—usually limited to small strains but can develop intermediate strain levels using softer or smaller diameter test specimens—covered in part by ASTM Standard D4015.

Cyclic Triaxial—Involves axial loading of solid cylinders—shear modulus and damping are not measured directly but are interpreted from Youngs modulus and damping—traditionally used at larger strains but can be extended to smaller strains with special equipment and instrumentation—covered in part by ASTM Standard D3999.

Cyclic Simple Shear—Loading conditions more closely approximate those in the field but it is more difficult to set up undisturbed samples—again used more at larger strains but can be extended to smaller strains with special equipment and instrumentation.

Cyclic Torsional Shear—Can be conducted using solid or hollow cylinders—hollow cylinders give greatest control over the stress state but make testing of undisturbed samples very difficult—perhaps the easiest configuration for testing over a very wide strain range.

Again, no single test is free of limitations and it is desirable to test the same soil in two or more kinds of apparatus as a cross-check. While it is traditional to present the results of these tests just as values of $G_{\max}$ and modulus reduction and damping curves, the stress-strain curves should also be recorded and presented even when the intent is to conduct only equivalent linear analyses since the stress-strain loops can provide important clues regarding the validity of the data. When nonlinear analyses are to be run it is even more critical to record and present the stress-strain loops and to track the development of any excess pore pressures.

7.A.1.3.3 Backcalculation from Observations of Ground Motion. An important third source of data on dynamic properties is provided by backcalculation from observations of ground motions at vertical arrays or close by soil/rock station pairs. This technique will not usually be practical as part of a specific site investigation unless the site is located in a highly seismic area, however, estimates of both the $G_{\max}$ profile and the modulus reduction and damping curves that can be obtained by backcalculation can provide important insights into the uncertainty associated with various field and laboratory measurement techniques if detailed investigations are performed at the same site. In non-seismic areas it is not possible to obtain data on the strain dependence of dynamic properties but observation of microtremors in vertical arrays can provide a cross-check on other measurements of the $G_{\max}$ or V_s profile.

7.A.1.4 Sources of Uncertainty

While it is very easy to make both random and systematic gross errors in both field and laboratory measurements, these can be minimized by: (1) using properly trained and experienced personnel; (2) following appropriate quality assurance procedures; and (3) always using two or more different procedures to obtain key measurements. However, there are additional sources of uncertainty which from a practical point of view often cannot be eliminated. It is of the greatest importance that the engineers involved in both the measurement of properties and their use in analyses understand the sources of these uncertainties so that they can work towards minimizing them and, where they cannot be reduced to the level at which they are no longer significant, accounting for them in conducting and interpreting the results of the analyses. These sources of uncertainty include:

7.A.1.4.1 Inherent Variability of Soil Deposits. When soil profiles are divided by geologists or engineers into layers that might be assigned short descriptions such as silty clay or gravelly sand, the soils within these layers are rarely homogeneous. This can be readily seen if the grain size curves or the Atterberg limits for a number of samples from one layer are plotted together.

Also changes of soil type can often be seen within the length of a sample that has been taken from within a broadly defined layer that might otherwise be assumed to be homogeneous. This is especially true if borings have been logged principally from cuttings. Ideally, small diameter continuous borings and/or cone soundings should be made prior to drilling larger diameter borings to obtain samples for more sophisticated laboratory tests, both to quantify the variability of the deposit and to optimize locations for sampling.

As a general rule, finer grained soils that have been deposited in a low-energy environment will be more uniform than coarser grained soils deposited in high-energy environments. Residual soils and other soils with variable cementation may also show high variability even though cuttings and samples appear to be uniform. Thus an understanding of the regional and site geology and depositional history is important to planning and executing studies of soil properties.

7.A.1.4.2 Inherent Anisotropy. Even soils which are apparently homogeneous usually exhibit at least some degree of inherent anisotropy as a result of the impact of their method of deposition and subsequent stress history on the soil fabric. Commonly, but not always, this leads to the strength and stiffness being lower for shearing on horizontal planes as opposed to vertical planes. Inherent anisotropy is one of the contributors to shear wave velocity varying with direction and to differences in behavior in different types of laboratory tests.

7.A.1.4.3 Anisotropic In Situ Stresses. Additional anisotropy of in situ soil properties can result from an anisotropic state of stress being superimposed on inherent anisotropy. Virtually all soils are subjected to an anisotropic state of stress in the field with the lateral effective stress being about half the vertical effective stress for normally consolidated soils but increasing with the degree of overconsolidation. Because the shear wave velocity is a function of the effective stresses only in the direction of wave propagation and the direction of particle motion, but not in the third direction (Roesler, 1979; Stokoe et al., 1985; Yan and Byrne, 1991), measured shear wave velocities may vary with the direction and polarity of the waves that are generated.

7.A.1.4.4 Drilling and Sampling Disturbance. While it is obvious that samples may suffer from disturbance as a result of poor drilling, sampling and sample handling practices, it is not so widely recognized that some disturbance is inevitable as a result of release of the in situ stresses. This problem becomes more severe with increasing depth and even if it is possible to establish the in situ state of stress and replicate it in the laboratory, it may not be possible to eliminate the effects of unloading and reloading the sample (see, for instance, Thomann and Hryciw, 1992). It is well established that the shear modulus tends to increase and the damping decrease with the time that samples are consolidated under sustained pressure in the laboratory and it has been suggested that extrapolation of laboratory results to the geologic age of the soil provides the best estimate of the in situ behavior (see, for instance, Anderson and Stokoe, 1978), but this procedure does not necessarily compensate correctly for the differences in the stress states in the field and in the laboratory and for the stress path followed in the sampling and reconsolidation process.

7.A.1.4.5 Limitations of Laboratory Equipment. No laboratory equipment is capable of fully representing the in situ stress state and deformation conditions including the three-dimensional nature of earthquake ground motion. Thus, all laboratory testing devices represent a compromise of one kind or another and it should not be surprising that there are some differences between results of cyclic loading tests performed in different kinds of apparatus (see, for instance, Pyke, 1978; Bhatia et al., 1985; Talesnick and Frydman; 1991).

7.A.1.5 Integration and Interpretation of Data

It should be apparent from the foregoing discussion that no data, be it acquired in the field or the laboratory, should automatically be used in analyses without some thought being given to the necessity for integrating it with data from other sources and making judgments as to the most appropriate values for use in analyses. Nonetheless, substantial progress has been made in the care with which both field and laboratory tests are conducted so that it is now generally possible to obtain reasonable agreement between the values of V_s or G_{max} measured in the laboratory and the field. Indeed, this is a basic check that should always be made to check that the samples as reconsolidated and tested in the laboratory are representative of field conditions. Further research and development work in this area is still required, however, as definitive guidance on the optimum procedures for reconsolidating samples is lacking, especially for overconsolidated soils. In this respect, it is worth noting that measurement of G_{max} in the field using both SH and SV waves and then checking G_{max} in the laboratory in both horizontal and vertical directions using "bender elements" (see, for instance, Dyvik and Madshus, 1985; Thomann and Hryciw, 1990), might provide a high level of confidence that field conditions have been satisfactorily re-established. Modulus reduction and damping curves obtained from such samples would then represent the best estimate that can be made short of the development of techniques for making measurements at large strains in the field, but these estimates would still need to be verified by cross-checking with forward or backcalculation of observed behavior at well instrumented field sites.

7.A.2 Sources of Literature

In common with many other fields of study, the most detailed information regarding both equipment and results is provided only in research reports and Ph.D. theses which might not be formally published. Even in these documents important details, particularly with regard to problems or ideas that might not have worked out, may not be included and it is invariably useful to contact the workers involved to discuss their published and on-going work. More formal publications include:

(1) Papers in the principal geotechnical engineering journals: *The Journal of Geotechnical Engineering of ASCE*, *The Geotechnical Testing Journal of ASTM*, *Géotechnique*, *The Canadian Geotechnical Journal*, and *Soils and Foundations*.

(2) The proceedings of two specialty conferences organized by the Geotechnical Engineering Division of the ASCE:

"Earthquake Engineering and Soil Dynamics," Pasadena, 1978.

"Earthquake Engineering and Soil Dynamics II, Recent Advances in Ground-Motion Evaluation," Park City, 1988.

(3) The proceedings of sessions sponsored by the Geotechnical Engineering Division at ASCE national conventions:

"Soil Sampling and Its Importance to Dynamic Laboratory Testing," Chicago, 1978.

"Measurement and Use of Shear Wave Velocity for Evaluating Dynamic Properties," Denver, 1985.

"Richart Commemorative Lectures," Detroit, 1985.

"Advances in the Art of Testing Soils Under Cyclic Conditions," Detroit, 1985.

"Recent Advances in Instrumentation, Data Acquisition and Testing in Soil Dynamics," Orlando, 1991.

(4) Special Technical Publications of the American Society for Testing and Materials:

"Dynamic Geotechnical Testing," STP No. 654, 1978.

"Soil Specimen Preparation for Laboratory Testing," STP 599, 1976.

"Laboratory Shear Strength of Soil," STP 740, 1981.

"Advanced Triaxial Testing of Soil and Rock," STP 977, 1988.

"Geophysical Applications for Geotechnical Investigations," STP 1101, 1990.

(5) The proceedings of two conferences organized by the University of Missouri, Rolla:

International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St Louis, 1981.

Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St Louis, 1991.

(6) The proceedings of two recent national and international conferences published by the Earthquake Engineering Research Institute:

Fourth U.S. National Conference on Earthquake Engineering, Palm Springs, California, May 20–24, 1990.

Fourth International Conference on Seismic Zonation, Stanford, California, August 25–29, 1991.

(7) The proceedings of other recent international conferences including:

International Conferences on Soil Mechanics and Foundation Engineering:

11th, San Francisco, August 1985.

12th, Rio de Janeiro, August 1989.

World Conference on Earthquake Engineering:

8th, San Francisco, June 1984.

9th, Tokyo, August 1988.

10th, Madrid, July 1992.

International Conferences on Soil Dynamics and Earthquake Engineering:

1st, Southampton, U.K., August 1983.

2nd, Aboard Queen Elizabeth 2, June 1985.

3rd, Princeton, New Jersey, June 1987.

4th, Mexico City, October 1989.

(8) Other special publications, including:

"Soils Under Cyclic and Transient Loading," Proceedings of the International Symposium on Soils Under Cyclic and Transient Loading, Swansea, A. A. Balkema, 1980.

"Earthquake Geotechnical Engineering," Proceedings of Discussion Session on Influence of Local Conditions on Seismic Response. Twelfth International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Japanese Society of Soil Mechanics and Foundation Engineering, 1989.

7.A.3 Previous Summary Papers

The first significant work on dynamic properties was conducted in the 1960's and much of the early work is summarized in two major reviews:

H. B. Seed, and I. M. Idriss, "Soil Moduli and Damping Factors for Dynamic Response Analyses," Report No. EERC 70-10, University of California, Berkeley, December 1970. (Also Section 5, "Soil Behavior Under Earthquake Loading Conditions," Report to U.S. Atomic Energy Commission, SW-AA, January 1972.)

Hardin, B. O., Drnevich, V. P., "Shear Modulus and Damping in Soils: Measurements and Parameters Effects," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 98, No. SM6, 1972, pp. 603-624.

Hardin, B. O., Drnevich, V. P., "Shear Modulus and Damping in Soils: Design Equations and Curves," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 98, No. GT7, 1972, pp. 667-692.

While the Hardin and Drnevich papers provided a more detailed treatment of the subject, giving expressions for the variation of shear modulus and damping with a number of parameters, the Seed and Idriss report, which gave typical ranges for modulus reduction and damping curves for sands and clays, was generally easier to follow and found very wide use in practice.

The Seed and Idriss report was subsequently extended to cover gravelly soils in the following paper:

Seed H. B., Wong, R. T., Idriss, I. M., Tokimatsu, K., "Moduli and Dynamic Factors for Dynamic Analyses of Cohesionless Soils," *Journal of Geotechnical Engineering*, ASCE, Vol. 112, No. 11, 1986, pp. 1016-1032.

In both the Seed and Idriss report and the Hardin and Drnevich papers the modulus reduction curves for clays fell more sharply than the curves for sand, however, it turns out that this early conclusion was in error and that clayey soils are generally more elastic (that is, they show a lower rate of decrease of secant shear modulus with increasing shear strain) than sands. Revised modulus reduction and damping curves for clays were thus presented by Seed and his co-workers and by Dobry and Vucetic as follows:

- Dobry, R, Vucetic, M., "Dynamic Properties and Seismic Response of Soft Clay Deposits," Proceedings of the International Symposium on Geotechnical Engineering of Soft Soils, Mexico City, 1987, Vol. 2, pp. 51-87.
- Sun, J. I., Golesorkhi, R., Seed, H. B., "Dynamic Moduli and Damping Ratios for Cohesive Soils," Report No. EERC-88/15, University of California, Berkeley, August 1988.
- Vucetic, M., Dobry, R., "Effects of Soil Plasticity on Cyclic Response," *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No.1, 1991, pp. 89-107.

While the preceding papers trace the evolution of practice in the United States of America, notable contributions have been made by a number of workers in Japan. Earlier work and significant new contributions were reported by Kokusho and his co-workers in the following papers covering sands, gravels and clays:

- Kokusho, T., "Cyclic Triaxial Test of Dynamic Soil Properties for Wide Strain Range," *Soils and Foundations*, Vol. 20, No. 2, 1980, pp. 45-60.
- Kokusho, T., Esashi, Y., "Cyclic Triaxial Test On Sands and Coarse Materials," Proceedings Tenth International Conference on Soil Mechanics and Foundation Engineering, Stockholm, 1981, Vol. 1, pp. 673-676.
- Kokusho, T., Yoshida, Y., Esashi, Y., "Dynamic Properties of Soft Clay For Wide Strain Range," *Soils and Foundations*, Vol. 22, No. 4, 1982, pp. 1-18.

Other reviews with more emphasis on the implementation of dynamic properties in analyses include:

- Richart, F. E., "Some Effects of Dynamic Soil Properties on Soil-Structure Interaction," *Journal of Geotechnical Engineering Division*, ASCE, Vol. 101, No. GT12, 1975, pp. 1197-1240.
- Wood, D. M., "Laboratory Investigations of the Behaviour of Soils under Cyclic Loading: A Review," *Soil Mechanics — Transient and Cyclic Loads*, Edited by G. N. Pande and O. C. Zienkiewicz, John Wiley & Sons, 1982.
- Ishihara, K., "Evaluation of Soil Properties for Use in Earthquake Response Analysis," *Geomechanical Modelling in Engineering Practice*, Edited by R. Dungar and J. A. Studer, A. A. Balkema, 1986.

Review papers emphasizing in situ measurements include:

- Woods, R. D., "Measurement of Dynamic Soil Properties," Earthquake Engineering and Soil Dynamics, Proceedings of the ASCE Geotechnical Engineering Specialty Conference, Pasadena, 1978, Vol. 1, pp. 91-178.
- Stokoe, K. H., "Field Measurement of Dynamic Soil Properties," Proceedings of the Second ASCE Conference on Civil Engineering and Nuclear Power, Knoxville, Tennessee, Vol. 2, pp. 1-31.
- Woods, R. D., "Field and Laboratory Determination of Soil Properties at Low and High Strains," Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, 1991, Vol. 2, pp. 1727-1741.

Review papers which emphasize integration of field and laboratory data and static and dynamic properties include:

- Jamiolkowski, M., et al., "New Developments in Field and Laboratory Testing of Soils," Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, Vol. 1, pp. 57-153, 1985.
- Jamiolkowski, M., Leroueil, S., LoPresti, D. C. F., "Design Parameters from Theory to Practice," Theme Lecture, Geo-Coast '91 Conference, Port and Harbour Research Institute, Yokohama, 1991.

More recent Japanese work which emphasize comparisons of field and laboratory data are summarized in:

Kokusho, T., "In Situ Dynamic Soil Properties and Their Evaluations," Proceedings of the Eighth Asian Regional Conference on Soil Mechanics and Foundation Engineering, 1987, Vol. 2, pp. 215-240.

Kokusho, T., "Seismic Response of Soil Layer and Its Dynamic Properties," Preprint, Tenth World Conference on Earthquake Engineering, Madrid, 1992.

And, a recent summary co-sponsored by the Electric Power Research Institute is published as:

Dobry, R., et al., "Low and High Strain Cyclic Material Properties," Proceedings NSF/EPRI Workshop on Dynamic Soil Properties and Site Characterization, Report No. NP-7337, Electric Power Research Institute, Palo Alto, Vol. 1, June 1991.

7.A.4 Summary of Factors Affecting Modulus Reduction and Damping Curves

It is clear from the preceding review that the primary variation in modulus reduction and damping curves is with soil type, coarser, cohesionless soils showing a greater reduction in modulus with increasing shear strain than fine-grained, cohesive soils. This variation is summarized in Figure 7.A-3 which is based on the publications of Seed and Idriss and their co-workers but is also consistent with the findings of many other workers whose contributions are cited in Sections 7.A.3 and 7.A.7.

The published data on damping are more confusing, in part because of measurement errors in much of the earlier work, so rather than showing an equivalent figure for damping based on published data, companion damping curves are presented in Section 7.A.6, following the description of a simple model that generates consistent modulus reduction and damping curves. The general trend indicated by the published data for damping at intermediate to high strains are, however, compatible with the trend shown for the modulus reduction factors, namely, that coarser, cohesionless soils show greater values of damping with increasing shear strain than fine-grained, cohesive soils.

After soil type, the most significant factors affecting modulus reduction and damping curves are confining pressure for cohesionless soils and plasticity (as indicated by the plasticity index, or P.I.) for cohesive soils. Selected data on the variation of modulus reduction factors and damping ratios with confining pressure are shown in Figures 7.A-4 and 7.A-5. These data suggest that laboratory tests on freshly deposited clean sands (which are sometimes referred to as "baby" sands) show a greater sensitivity to confining pressure than natural soils but that there is still a systematic variation of modulus reduction factors and damping ratios with confining pressure for natural silty and clayey sands. Typical variations of the modulus reduction factors and damping ratios with P.I. for cohesive soils are shown by Sun et al. (1988) and Vucetic and Dobry (1991) and in Section 7.A.6. Since the shear modulus decreases more slowly with increasing shear strain for soils with a higher P.I., a confusion in terminology arises since more "plastic" soils in fact show more "elastic" stress-strain behavior.

Damping at low strains does not, however, follow quite the same pattern as modulus reduction factors and damping at higher strains. While low strain damping decreases with increasing confining pressure as shown in Figure 7.A-6, data published by Kokusho (1982) and some of the data developed in this study suggest that for fine-grained soils, while damping at high strains decreases with increasing P.I., low strain damping increases with increasing P.I. These variations are relatively minor, however, compared to what appears to be a major increase with either the grain size or the uniformity of the soil. This increase in low strain damping for coarser or less homogeneous soils is greater than might be inferred from their more nonlinear modulus reduction curves and smaller reference strains and comparisons of field and laboratory measurements of damping that are reported in Section 8 indicate that such soils may exhibit even greater damping in the field. Such differences may result in part from the fact that field measurements are typically made at somewhat higher frequencies than laboratory measurements, but may also result from back-scattering and other wave-propagation effects which cause the apparent damping in the field to be greater than the "intrinsic" damping which is measured using relatively homogeneous samples in the laboratory.

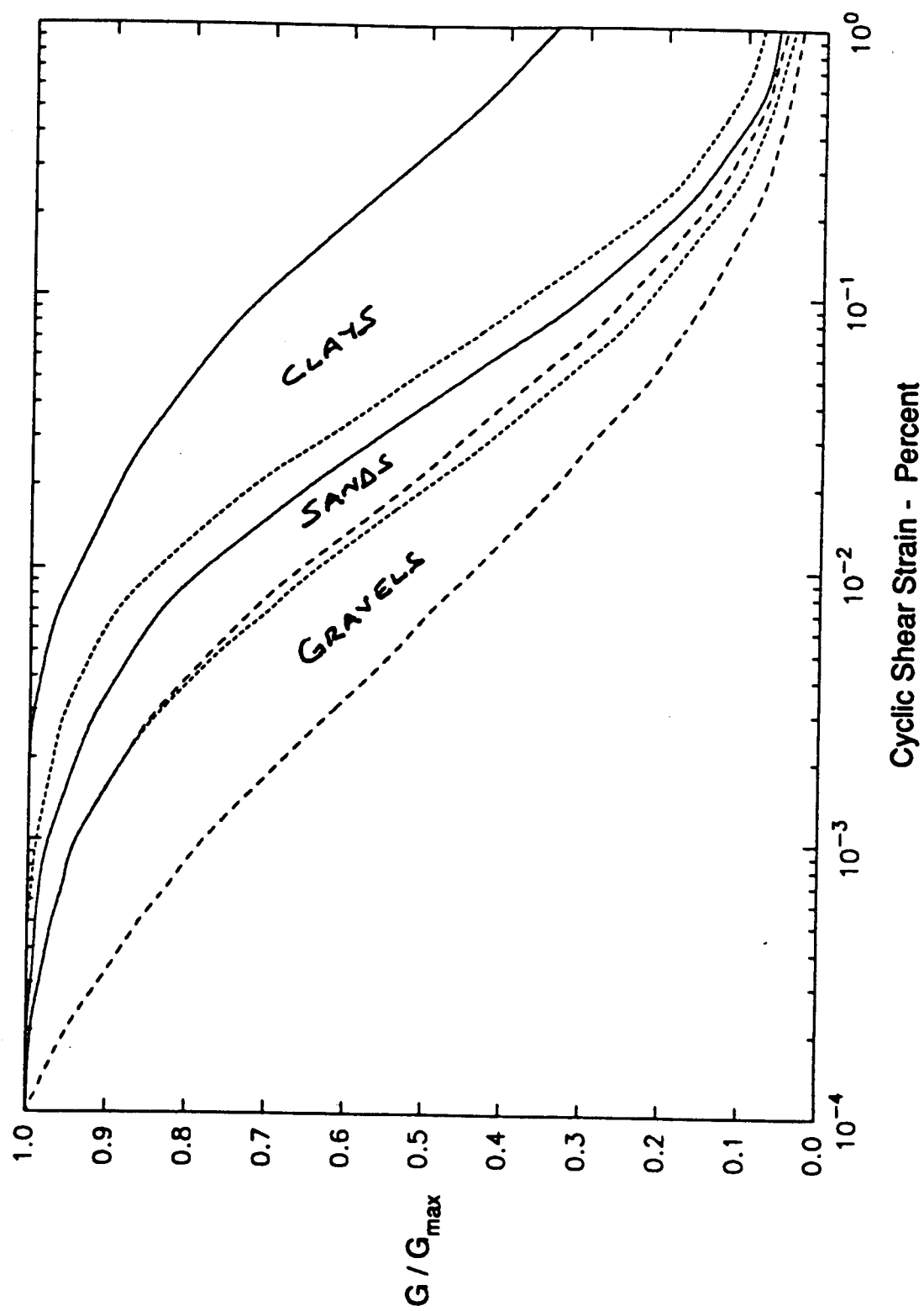


Figure 7.A-3
Typical Ranges for Modulus Reduction Curves

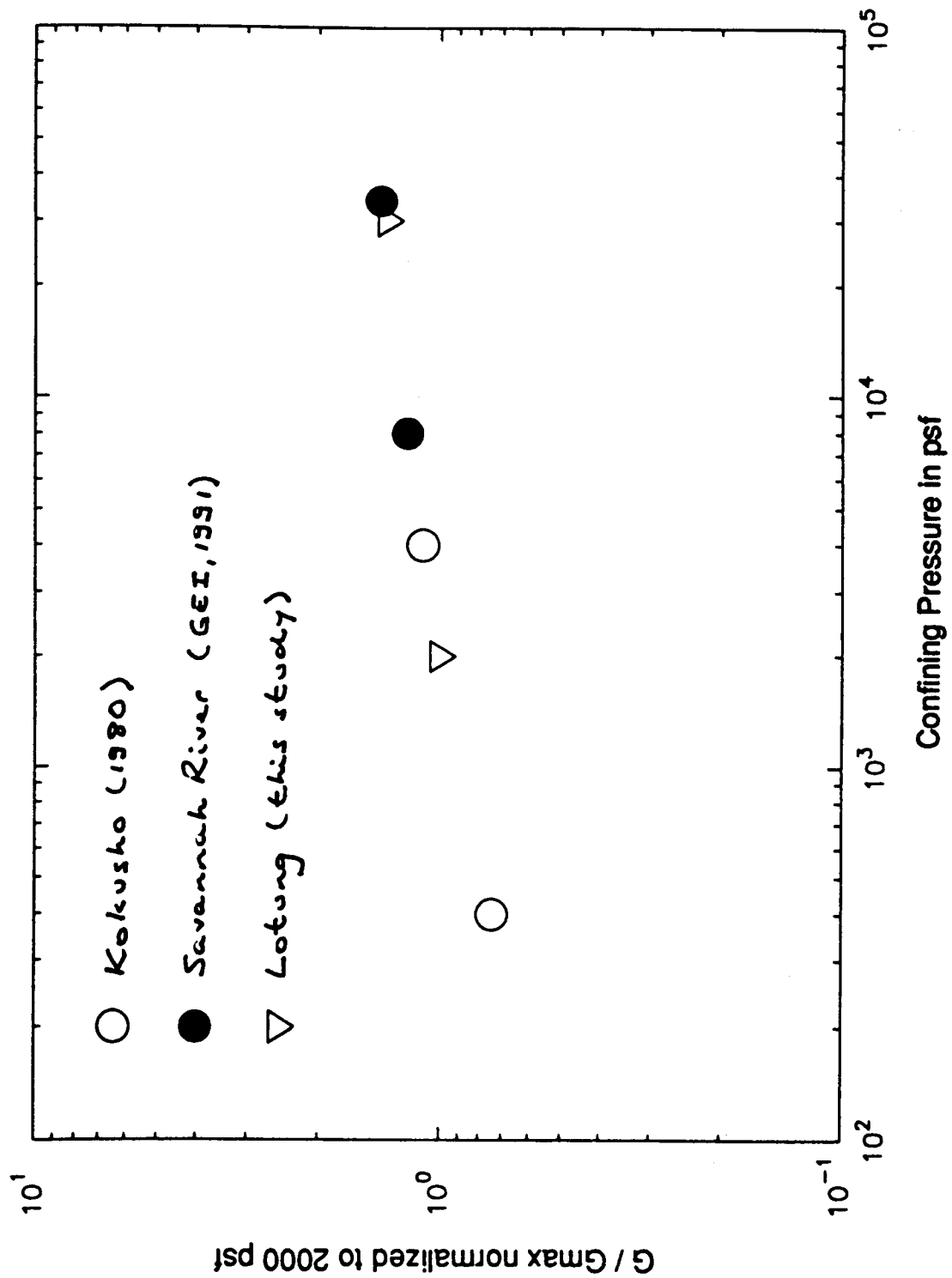


Figure 7.A-4
Variation of Modulus Reduction Factors with Confining Pressure

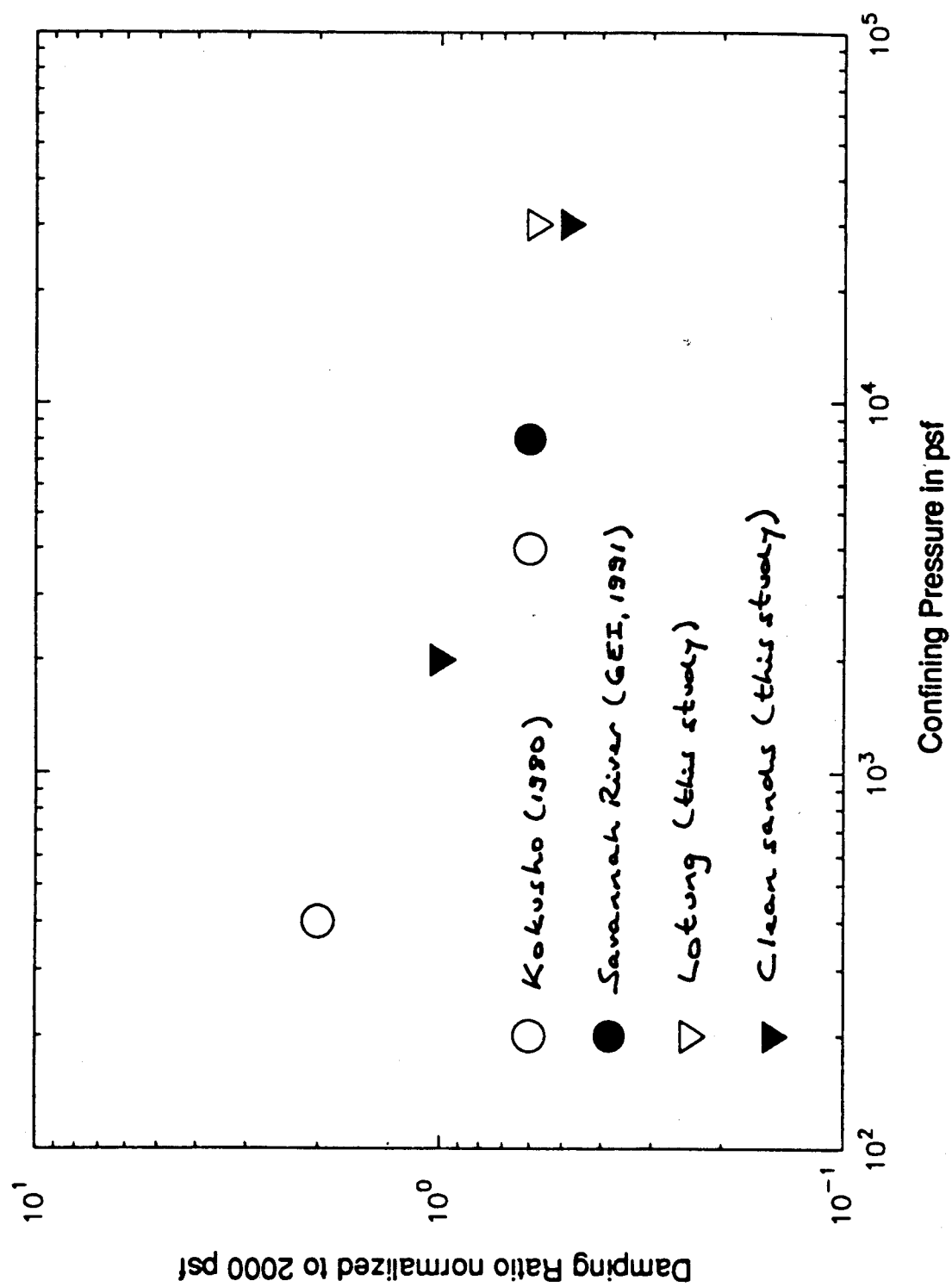


Figure 7.A-5
Variation of Damping Ratio with Confining Pressure

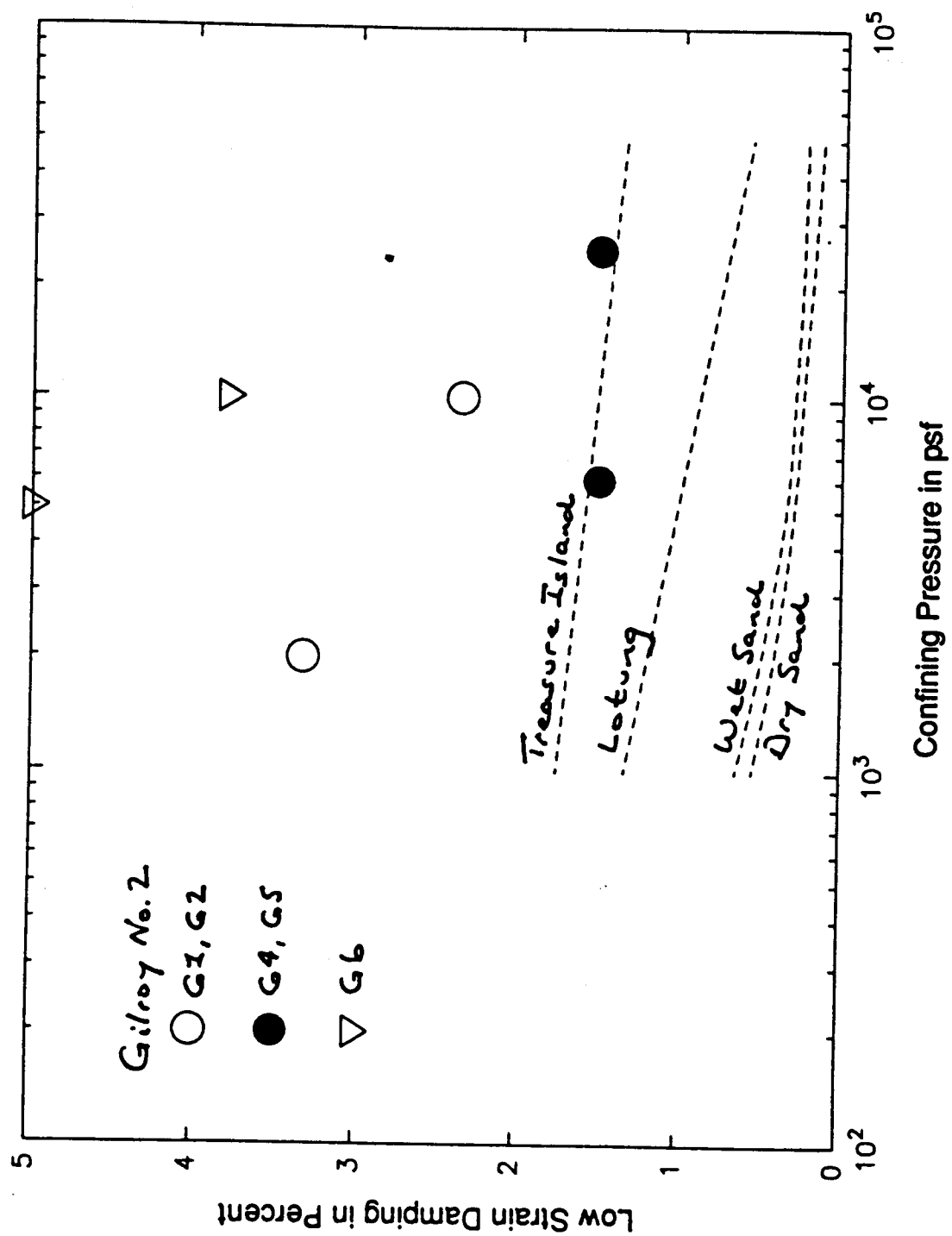


Figure 7.A-6
Variation of Low Strain Damping with Confining Pressure

In comparison with basic soil type and confining pressure or plasticity, all other factors appear to be secondary in nature (see, for instance, Iwasaki and Tatsuoka, 1977; Tatsuoka et al., 1979; Yu and Richart, 1984; Alarcon et al., 1989) although in some special situations these other factors may become important. For instance, the effect of the frequency of loading is generally not important over the range of frequencies used in laboratory tests but may become important when comparing the results of laboratory tests with field tests, as noted above. Rate of loading effects also appear to be significant in generating damping at low strains as discussed in the following section. As discussed in Section 7.A.6, the effect of overconsolidation on modulus reduction and damping curves may be obscured by other factors and deserves further study.

7.A.5 An Improved Nonlinear Soil Model

7.A.5.1 Introduction

A simple hyperbolic model of nonlinear shear stress-shear strain behavior is developed in this section in order to: (1) provide a basis for generating consistent modulus reduction and damping curves; (2) study a possible mechanism for including low strain damping in such models; and (3) study mechanisms for modeling the degradation and degeneration of the shape of stress-strain loops which may significantly affect the damping at higher strain levels. This model is a one-dimensional model in the sense that it only attempts to model the shear stress-shear strain relationship and does not attempt to model coupled shear and compression behavior under generalized three-dimensional loadings. Examples of more complete three-dimensional models are given by Prevost (1989) and Li et al. (1992).

7.A.5.2 Simple Hyperbolic Model

A simple model of soil behavior under simple shear loading conditions can be constructed by assuming that the shape of the shear stress-shear strain relationship can be represented by a hyperbola which has an initial slope equal to $G_{\max}$, the shear modulus at small strains, and is asymptotic to $\tau_{\max}$, the "shear strength". The quantity

$$\gamma_r = \frac{\tau_{\max}}{G_{\max}} \quad 7.A-1$$

defined by Hardin and Drnevich (1972) as the reference strain turns out to be useful both in expressing the stress-strain relationship in mathematical form and as a measure of the relative values of $G_{\max}$ and $\tau_{\max}$. Soils with larger values of γ_r have greater "shear strengths" relative to their small strain modulus and show more elastic stress-strain behavior than soils with smaller values of γ_r . Thus gravelly soils have low values of γ_r and more plastic clays have high values.

The quantity $\tau_{\max}$ is referred to in this discussion as the "shear strength", using quotation marks to indicate that while the asymptote of the stress-strain relationship should be related to the shear strength of the soil under the appropriate loading conditions, it is in fact very difficult to measure the shear strength under the right loading conditions and, further, the stress-strain relationship for many soils may deviate from a true hyperbolic shape at larger strains so that a better fit to observed stress-strain relationships at small to moderate strains can be obtained by treating $\tau_{\max}$ as a fitting parameter, rather than deriving it independently as the shear strength.

The quantity $G_{\max}$ is best obtained by careful measurement of the shear wave velocity profile at the site in question but for parametric studies it may be useful to study the variation of $G_{\max}$ with confining pressure using either of two standard formula. One such formula due to Hardin (1978) is an update of the earlier expression given by Hardin and Black (1968). The revised expression is:

$$G_{\max} = A p_a F(e) OCR^k \left(\frac{\sigma'_m}{p_a} \right)^n \quad 7.A-2$$

Where:

- e = void ratio
- p_a = atmospheric pressure
- σ'_m = mean effective stress
- n = an exponent which has a value in the order of 0.5
- OCR = overconsolidation ratio
- k = an exponent whose value varies from 0 if the P.I. is zero to 0.5 if the P.I. equals 100 or more and
- $F(e) = (0.3 + 0.7e^2)^{-1}$

Hardin suggested a constant value of 625 for the parameter A but values ranging from 400 to 700 are found in practice.

An alternate expression can be obtained by putting the expression given by Seed and Idriss (1970) in non-dimensional form as follows:

$$G_{\max} = K_G p_a \left(\frac{\sigma'_m}{p_a} \right)^{ng} \text{OCR}^{nocr} \quad 7.A-3$$

Where:

- K_G = a dimensionless parameter equivalent to the $K_{2\max}$ of Seed and Idriss
- p_a = atmospheric pressure
- σ'_m = mean effective stress
- ng = an exponent
- OCR = overconsolidation ratio
- $nocr$ = a further exponent similar to Hardin's k

Typical values of the parameter K_G are shown in Figure 7.A-7. The exponent, ng , is commonly taken to be equal to 0.5 for cohesionless soils but this value and the use of the mean effective stress are in fact simplifications. Careful measurements of shear wave velocities under controlled conditions in the laboratory suggest that the shear wave velocity is in fact a function of only the normal stress in the direction of wave propagation and the normal stress in the direction of particle motion. The shear modulus turns out to a function of the product of these two stresses both raised to something like the 0.25 power, so that use of the mean stress raised to the 0.5 power, where the mean stress is defined as $\sigma'_v(1 + k_0)/2$, is a good approximation for many conditions. For cohesionless soils the exponent ng increases at low confining pressures (below 1000 psf) and for cohesive soils the exponent approaches unity.

In older work the shear modulus of cohesive soils has often been reported in terms of the undrained shear strength, S_u , as $G_{\max}/S_u$, but this introduces the significant difficulties involved in measuring S_u and it is preferable to treat data from all soils in a consistent fashion as a function of the effective stresses and consolidation history.

The effect of overconsolidation on the shear modulus at small strains is in fact unclear. Jamiolkowski (personal communication, 1992) is adamant that $G_{\max}$ is independent of mechanical overconsolidation if the parameter K_G is made a function of the void ratio as is done in the Hardin expression but contrary evidence can also be found. In any event, it is clear from a number of studies (for instance, Weiler, 1988) that $G_{\max}$ does not increase as rapidly with increasing OCR as does the undrained shear strength, so that ratio $G_{\max}/S_u$ decreases with increasing OCR and the reference strain, which is proportional to the inverse of $G_{\max}/S_u$, must tend to increase with increasing OCR.

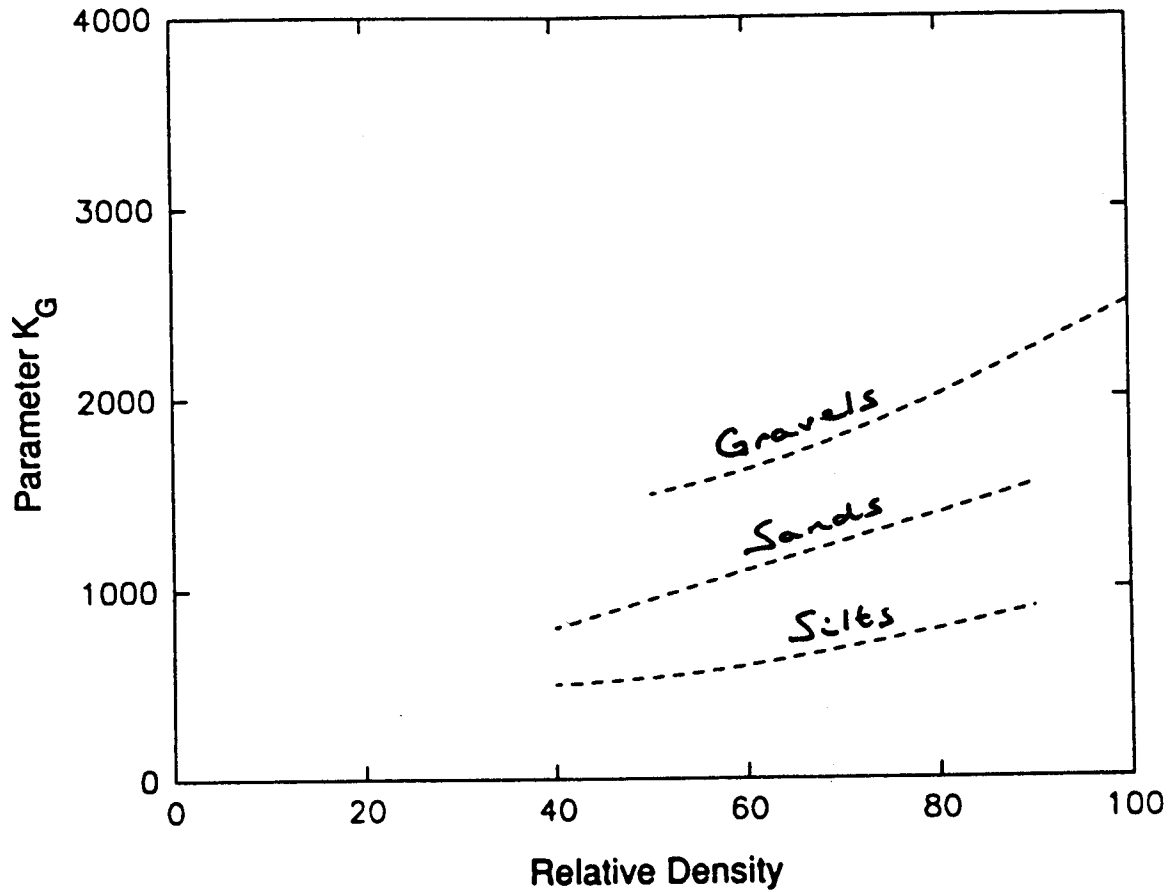


Figure 7.A-7
Variation of Parameter K_G with Soil Type

While the quantity $\tau_{\max}$ may best be viewed as a fitting parameter, it is convenient for modeling purposes to use the conventional Mohr-Coulomb failure criterion to express $\tau_{\max}$ as a function of the vertical effective stress:

$$\tau_{\max} = c + \sigma'_v \tan \phi \quad 7.A-4$$

Where:

- c = cohesion
- ϕ = angle of friction
- σ'_v = vertical effective stress

The angle of friction tends to decrease with increasing confining pressure and it is sometimes convenient to calculate ϕ as:

$$\phi = \phi_1 - \Delta\phi \log \left(\frac{\sigma_v}{p_a} \right) \quad 7.A-5$$

Where:

- ϕ_1 = angle of friction at one atmosphere of pressure
 $\Delta\phi$ = reduction in angle of friction per log cycle of pressure

Given appropriate values of $G_{\max}$ and $\tau_{\max}$ and hence the reference strain, γ_r , the relationship between shear stress and shear strain for initial loading can be written as:

$$\tau = \tau_{\max} \left(\frac{\frac{\gamma}{\gamma_r}}{1 + \frac{\gamma}{\gamma_r}} \right) \quad 7.A-6$$

and a simple model for cyclic loading can be obtained by applying the Cundall-Pyke hypothesis (Pyke, 1979) in which the shear stress following a load reversal is given by:

$$\tau = \tau_c + \tau_{\max} \left(\frac{\frac{\gamma - \gamma_c}{\gamma_r}}{1 + \frac{|\gamma - \gamma_c|}{c\gamma_r}} \right) \quad 7.A-7$$

in which:

- τ_c = shear stress at last reversal
 γ_c = shear strain at last reversal
and:

$$c = \left| \pm 1 - \frac{\tau_c}{\tau_{\max}} \right| \quad 7.A-8$$

where the first term is negative for unloading and positive for reloading. The factor c therefore has values which range from a minimum of 0 to a maximum of 2, as opposed to the alternate Masing hypothesis which uses a constant factor of 2. The stress-strain loops that are obtained using the Cundall-Pyke hypothesis are more realistic in several respects than those obtained using the Masing hypothesis and the resulting model allows the development of permanent strains which the Masing hypothesis does not.

It should be emphasized that the use of a hyperbolic shape is an approximation and a convenience. Other, more complex shapes could be adopted and cycled in accordance with the Cundall-Pyke hypothesis but a hyperbolic shape provides a good fit to much laboratory data on the variation of the secant, or equivalent linear, shear modulus with shear strain up to cyclic shear strains of 0.1 to 1 percent, beyond which the use of equivalent linear procedures is questionable and more elaborate models are required for nonlinear analyses. If the hyperbola is fitted to the relationship between secant shear modulus and cyclic shear strain after a given number of cycles of loading, this can be used both directly in equivalent linear analyses and as a basis for constructing a nonlinear soil model.

For use in equivalent linear analyses the basic hyperbolic relationship can be re-written as:

$$\frac{G}{G_{\max}} = \frac{1}{1 + \frac{\gamma}{\gamma_r}} \quad 7.A-9$$

which indicates that the reduction of secant shear modulus with increasing shear strain is solely a function of the reference strain. Thus, soils which show larger values for the reference strain, such as sands under higher confining pressures, more plastic clays and overconsolidated soils in general, show relatively less reduction of $G_{\max}$ with increasing cyclic shear strain.

7.A.5.3 Modeling of Low Strain Damping

While relationships between the variation of modulus with shear strain and the variation of damping with shear strain have commonly been developed and presented as if the modulus and damping are independent quantities, this cannot in fact be true if damping in soils is purely hysteretic, as is also commonly believed. As discussed in Section 7.A.1, if the damping in soils is purely hysteretic, then any model that describes the shear stress-shear strain relationship must also define the area of the stress-strain loops and hence the damping ratio. However, this creates a conflict with observed behavior in that even carefully calibrated laboratory measurements show finite values of damping at low strains where the secant modulus is constant. This observation has sometimes led to the suggestion that there may be small amounts of viscous damping in soils which are masked by hysteretic damping at larger strains, but that conclusion is not supported by measurements which show that small as well as large strain damping is largely independent of the frequency of loading. While Joyner et al. (1981) suggested a scheme for including frequency independent damping in nonlinear analyses, other nonlinear analysis schemes have usually included small amounts of viscous damping in an attempt to approximate the observed low strain damping at the frequency of greatest interest.

It turns out, however, that the low strain damping observed in the laboratory can be explained by including rate-of-loading effects in the stress-strain model in conjunction with recognizing that in most laboratory tests cyclic loads are applied using a sinusoidal loading function under which the rate of strain is markedly different at zero-crossings and at the ends of the loops. This point was originally made by Vucetic (1986) who suggested that it may explain why the ends of stress-strain loops are typically somewhat rounded. Thus, even at very low strains, the difference in the value of $G_{\max}$ at zero-crossings and at the ends of the loops may be sufficient to introduce hysteresis. Because the extreme points on the stress-strain loops are reached at a constant, infinitesimally slow, rate-of-strain, regardless of the frequency of the cyclic loading, there may not, however, be a significant change in the secant modulus.

Such a rate-of-strain effect can be introduced by defining a strain rate parameter, v_G , where, as suggested by Isenhower and Stokoe (1981), v_G equals the change in modulus per ten-fold increase in the strain rate normalized to the modulus at a shear strain rate of 0.01 percent per second. Then:

$$G_{\max}^* = G_{\max} \delta_G \delta_G^* \left(1 + v_G \log_{10} \frac{\dot{\gamma}}{0.0001} \right) \quad 7.A-10$$

where $\dot{\gamma}$ = shear strain (absolute) per second. A similar parameter, v_τ , can be defined to quantify rate-of-loading effects on $\tau_{\max}$.

Existing data on strain rate effects on shear modulus are limited, but data presented by Isenhower and Stokoe suggest that values of v_G in the order of 4 percent may be appropriate for San Francisco Bay Mud. Assuming for the moment that v_τ equals v_G , this value yields a low strain damping value of about 2 percent which is in the right order for Bay Mud. Because the low strain damping generated by the model invariably turns out to be equal to about half the value of the strain rate parameter over the range of interest, it is not in fact necessary to measure the rate of loading effect directly. If the appropriate value of low strain damping is known, the strain rate parameter can easily be set to obtain the desired damping. The stress-strain relationships and the modulus reduction and damping curves obtained using this approach are relatively insensitive to the frequency of loading and to the value assumed for v_τ . While further study of the strain-rate effect on both $G_{\max}$ and $\tau_{\max}$ is required, the preliminary results obtained in this study appear to offer a rational explanation of both the observation of rounded ends on stress-strain loops obtained using sinusoidal loadings and finite values of damping at low strains. Further, because available data suggest that strain-rate effects increase with increasing plasticity, this approach offers an explanation of why low strain damping may increase with increasing P.I. while damping at higher strains decreases with increasing P.I.

7.A.5.4 Modeling of High Strain Damping

One of the reasons that a simple hyperbolic model may no longer be adequate at large cyclic shear strains is that for most natural soils the shape of shear stress-shear strain loops for saturated soils tends to both degrade (flatten) and degenerate (change shape) with increasing number of cycles of loading. It turns out, however, that a good fit to observed shear stress-shear strain loops can still be obtained using a simple hyperbola as the backbone curve if $G_{\max}$ and $\tau_{\max}$ are multiplied by degradation and degeneration indices and updated values of $G_{\max}$ and $\tau_{\max}$ are calculated as:

$$G_{\max}^* = G_{\max} \delta_G \delta_G^* \quad 7.A-11$$

$$\tau_{\max}^* = \tau_{\max} \delta_\tau \delta_\tau^* \quad 7.A-12$$

where δ_G is the degradation index as defined by Idriss et al. (1978), δ_G^* is a multiplier that operates within each cycle of loading, and δ_τ and δ_τ^* are similar multipliers operating on $\tau_{\max}$. These degradation and degeneration indices must somehow be made a function of the amplitude and number of cycles of loading and can, if desired, be made a function of the sustained and transient excess pore pressures. Examples of possible functions for modeling the degradation and degeneration of stress-strain loops for clays using a total stress approach and for sands using an effective stress approach are given below.

7.A.5.5 Total Stress Modeling of Degradation and Degeneration

The degradation index, δ , as used by Idriss et al. (1978) is a measure of the degradation of the secant shear modulus with the number of cycles of loading at a given cyclic shear strain.

$$\delta = \frac{G_n}{G_1} \quad 7.A-13$$

Where:

G_1 = secant shear modulus on first cycle
 G_n = secant shear modulus on the n^{th} cycle

For irregular cyclic loadings the degradation index after each half cycle of loading is given by:

$$d_n = d_{n-1} \left\{ 1 + 0.5(d_{n-1})^{1/t} \right\}^{-t} \quad 7.A-14$$

Where:

δ = the present value of δ
 δ_{n-1} = the previous value of δ
 t = a parameter determined experimentally which describes the variation of δ with the number of cycles of loading at a given cyclic shear strain. Typical values of the parameter, t , as a function of P.I. are given by Vucetic (1992).

While the degradation index, δ , was defined by Idriss et al. to be a measure of the degradation of the secant modulus, it can be applied to both $G_{\max}$, the shear modulus at small strains, and $\tau_{\max}$, the shear strength in order to degrade the stress-strain relationship at an appropriate rate. Ideally, however, the degradation of $G_{\max}$ and $\tau_{\max}$ might be monitored separately in laboratory tests and different degradation indices would be applied to each of these quantities. Moreover, laboratory data sometimes indicate that the tangent shear modulus at the end of each stress-strain loop degrades little, if at all, but the shape of the hysteresis loops degenerates and the tangent modulus within each loop drops to a value

rather lower than the secant modulus measured through the ends of the loops. For these materials, more realistic stress-strain loops are obtained if $G_{\max}$, and $\tau_{\max}$, are reduced as a function of a modified degradation index, δ^* , which is dependent on the strain level:

$$\delta^* = 1 - (1 - \delta) \left[1 - \left(\frac{|\gamma - \gamma_0|}{g\gamma_r} \right)^e \right] \quad 7.A-15$$

Where:

- γ_0 = the average of the shear strains at the last two zero-crossings of the shear stress
- e = a fitting parameter which controls the rate at which degradation is activated within each cycle
- g = a fitting parameter which determines the shear strain at which degradation is no longer effective

Typical shear stress-shear strain loops generated using such a model are shown in Figure 7.A-8 for four different values of P.I., selecting values of $G_{\max}$ and $\tau_{\max}$ to give appropriate values of γ_r . Note that the example for a P.I. of 10 shows some degeneration of the shape of the loops which will impact the damping at higher strains and approaches the behavior of a cohesionless soil.

7.A.5.6 Effective Stress Modeling of Degradation and Degeneration

A simple scheme suggested by Seed, Martin and Lysmer (1976) to compute the excess pore pressures that may develop in saturated cohesionless materials when they are subjected to cyclic loading under undrained conditions may be used to model degradation and degeneration of shear stress-shear strain loops for cohesionless soils. Using this scheme, the excess pore pressures are computed at the end of each half-cycle and, if the low strain shear modulus and/or the shear strength have been specified to be a function of the mean effective stress, they will be recomputed at the end of each half-cycle.

The Seed et al. procedure for computing excess pore pressures relies on the fact that the curve of normalized excess pore pressure, u/σ'_m versus the normalized number of cycles of loading, N/N_L , where N_L is the number of cycles causing initial liquefaction or failure, has a characteristic shape as shown in the Seed et al. paper.

The number of cycles, N , or the number of cycles to initial liquefaction or failure, N_L , is always the number of uniform cycles of loading in laboratory tests and N_L is not constant but varies with the amplitude of cyclic shear stress that is applied. There are in fact any number of pairs of values N_L and τ_{av} , the uniform or average cyclic shear stress, which define the loading conditions causing initial liquefaction or failure. Both N_L and τ_{av} must therefore be defined for each element or layer. Normally N_L will be set to one value and τ_{av} will be computed for each element as a function of the initial mean effective stress.

Because the actual history of shear stress in any element is irregular, the equivalent number of uniform cycles with an amplitude of τ_{av} must be computed in order to use the curve of excess pore pressure. This is done by saving the peak shear stress in each half cycle of the shear stress defined by successive zero-crossings and using the weighing procedure described by Seed et al. (1975). This weighting procedure requires use of a curve based on the shape of the τ_{av} versus N_L line which defines initial liquefaction or failure, adjusted so that a cycle of stress with an amplitude equal to τ_{av} is given a weight of unity. Cycles with larger or smaller amplitudes are then equivalent to more or less than one cycle of the uniform or average cyclic shear stress.

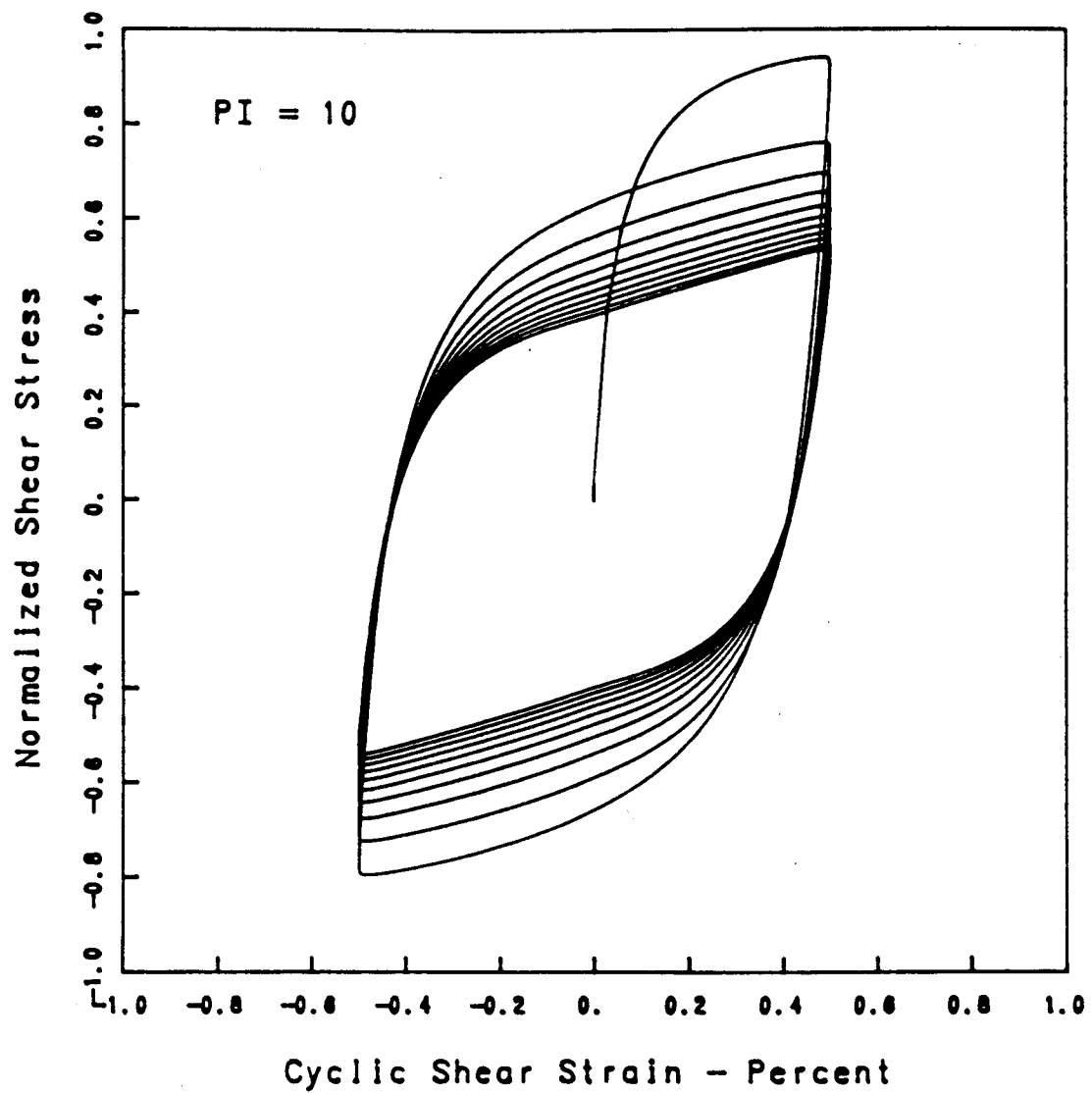


Figure 7.A-8a
Typical Stress-Strain Loops for Clay

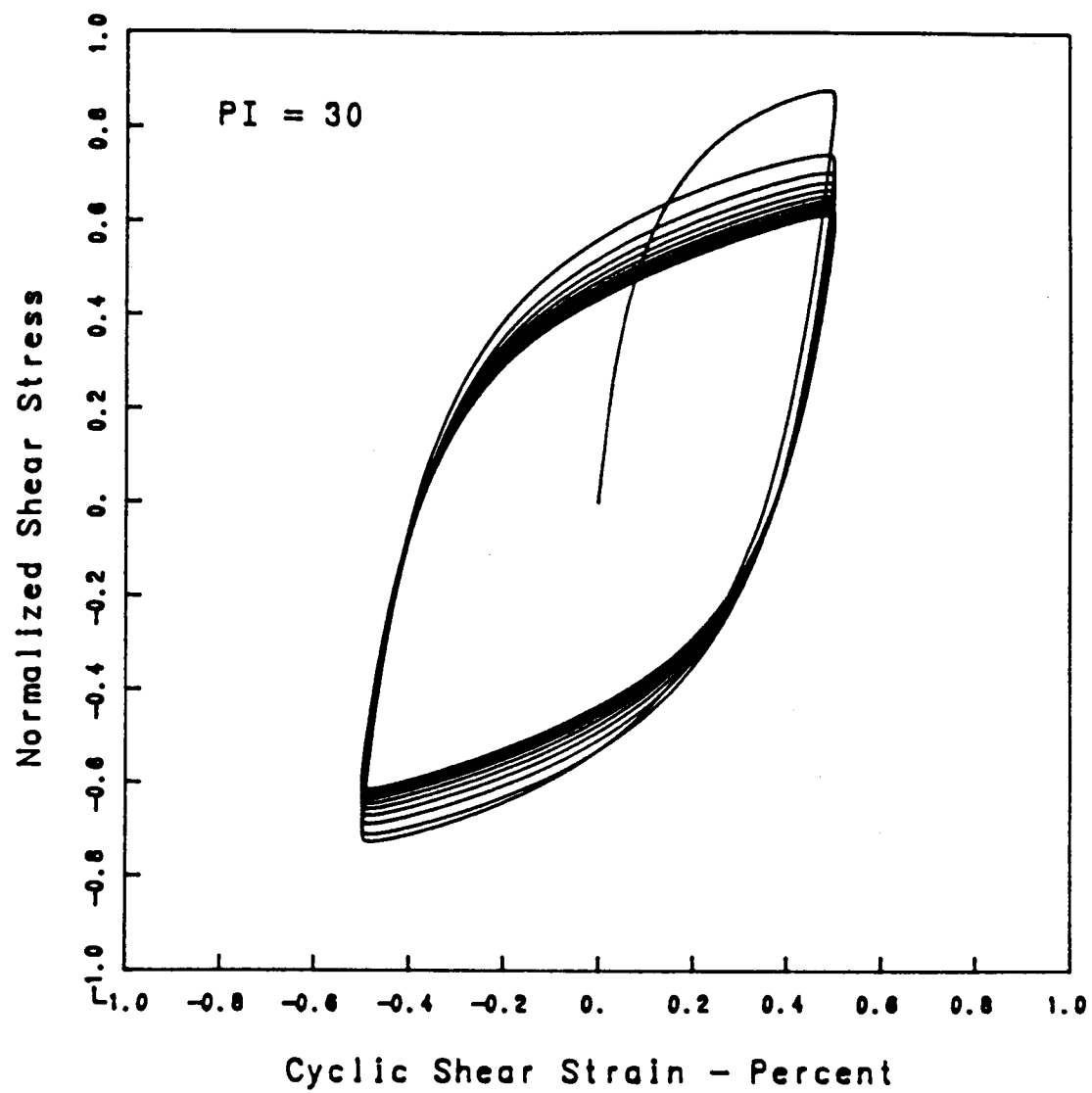


Figure 7.A-8b
Typical Stress-Strain Loops for Clay

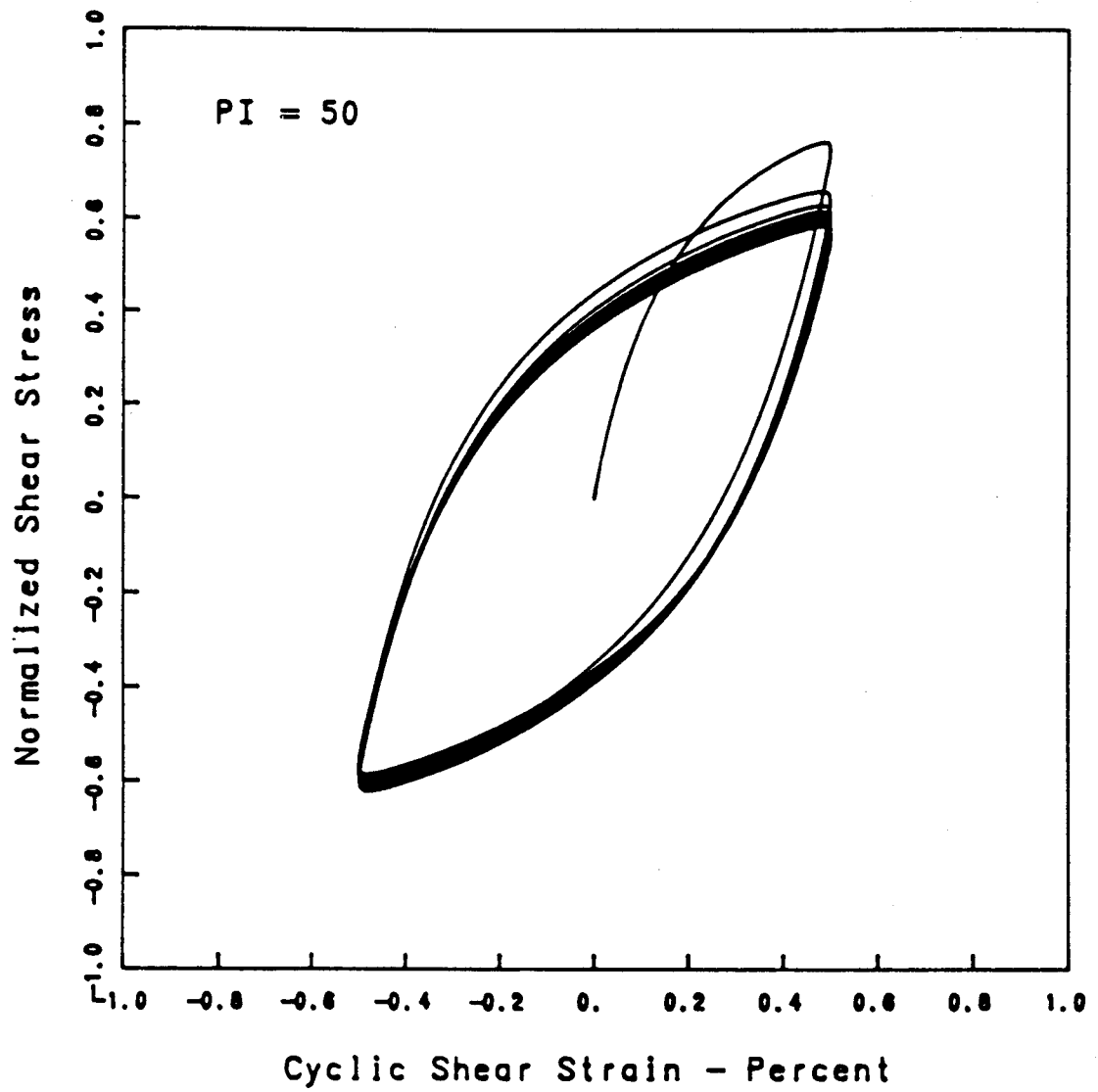


Figure 7.A-8c
Typical Stress-Strain Loops for Clay

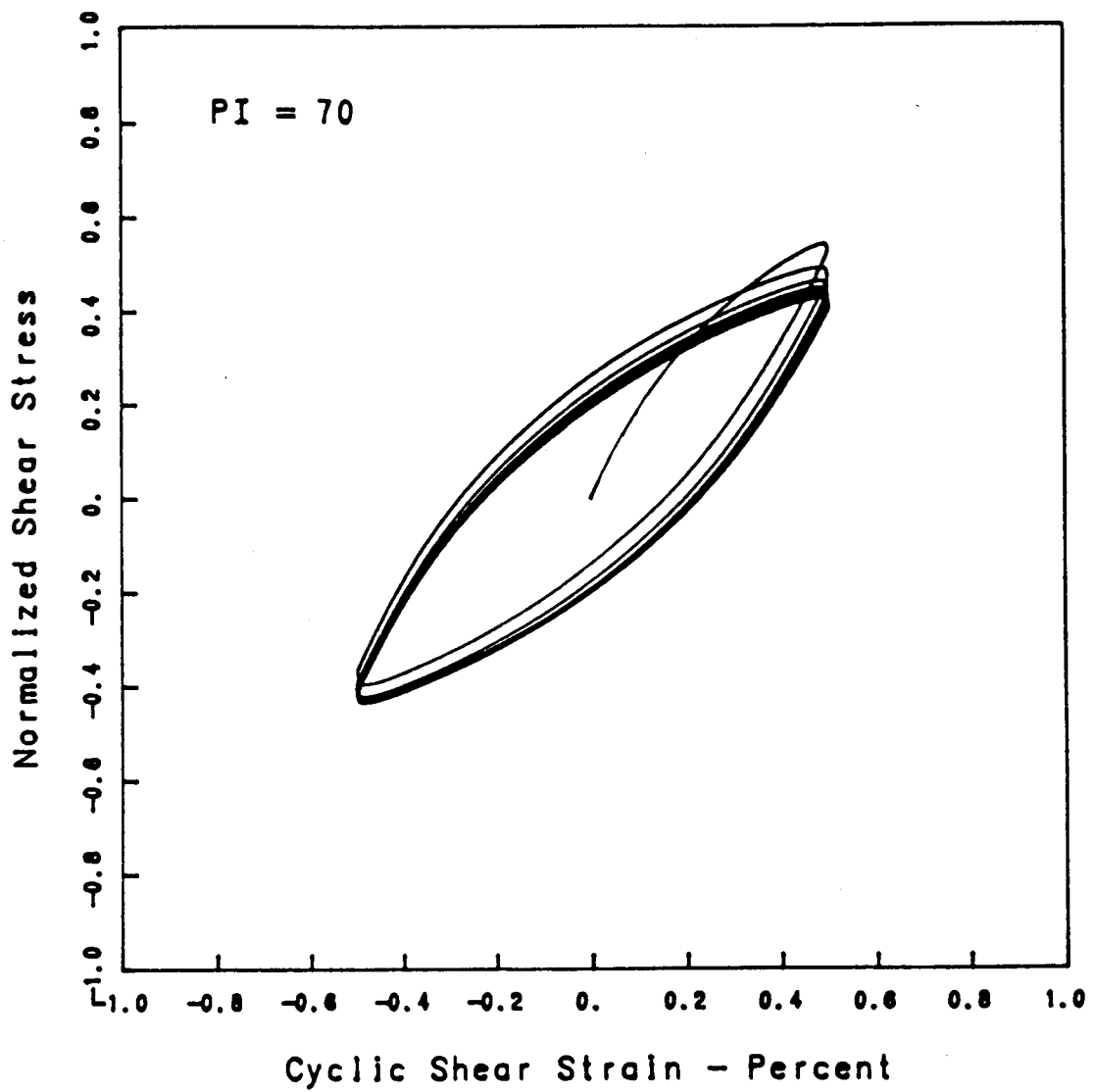


Figure 7.A-8d
Typical Stress-Strain Loops for Clay

As noted previously, when the use of pore pressure generation curves is specified in conjunction with stress-dependent values for $G_{\max}$ and $\tau_{\max}$, the values of $G_{\max}$ and $\tau_{\max}$ will be recomputed each half-cycle as a function of the current value of the excess pore pressure. Note that half-cycles are defined by crossings of the zero shear strength axis and that the excess pore pressures computed by the Seed et al. procedure are the peak or sustained excess pore pressures. Additionally, there will be transient fluctuations of the excess pore pressures within each cycle which increase as the relative density increases and the confining pressure decreases. These transient decreases in excess pore pressures within each half-cycle are what cause the characteristic degenerated shape of the stress-strain relationship obtained for silts and sands subjected to undrained cyclic loading.

The transient fluctuations of the excess pore pressures can be modelled using three additional parameters, e , f , and g , where the e and g parameters are the same as that used in the simple degradation model—that is, they control the sharpness and width of the transient decreases in pore pressure within each half-cycle—and f is the ratio of the transient excess pore pressure to the sustained excess pore pressure.

The shear stress-shear strain loops obtained using such a model for a typical saturated sand at three different confining pressures subjected to a cyclic shear strain of 0.5 percent are shown in Figure 7.A-9.

7.A.6 Standard Modulus Reduction and Damping Curves

The simple model developed in the previous section can now be used to develop modulus reduction and damping curves that are compatible with each other and consistent with the better published data.

Figures 7.A-10 and 7.A-11 show modulus reduction and damping curves for a wide range of soils in terms of the reference strain, γ_r . While the reference strain is not a very familiar index of soil behavior, it turns out to be a very appropriate index for this purpose and it is not difficult to gain a sense of the relationship between reference strain and soil type. Gravelly soils have reference strains as low as 0.01 percent and highly plastic clays have reference strains approaching 1.0 percent. Sandy soils at moderate confining pressures typically have reference strains in the order of 0.1 percent and it turns out that the modulus reduction and damping curves for 0.1 percent that are shown in Figures 7.A-10 and 7.A-11 are similar to the upper bound of the modulus reduction curves and the lower bound of the damping curves given by Seed and Idriss (1970), which have been found to be generally representative of many silty and clayey sands (I.M. Idriss, personal communication, 1992). It might be noted that the families of curves shown in Figures 7.A-10 and 7.A-11 generally show more linear behavior than was the case in the earliest published studies and that this observation is especially applicable to clayey soils.

The sensitivity of the modulus reduction and damping curves for dry or moist sands to the initial confining pressure is shown in Figures 7.A-12 and 7.A-13. Freshly deposited clean sands may show greater variation with confining pressure than that shown, and silty or clayey sands may show less variation.

Corresponding curves for saturated sands are shown in Figures 7.A-14 and 7.A-15. While there are only minor differences in the modulus reduction factors, note that the damping at high strains is markedly reduced for saturated sands, consistent with the experimental data of Matasovic and Vucetic (1992).

Modulus reduction and damping curves for clays are shown in Figures 7.A-16 and 7.A-17 as a function of the plasticity index. These curves are generally consistent with the curves of both Seed et al. (1988) and Vucetic and Dobry (1991).

Finally, modulus reduction and damping curves suitable for generic site response studies in Eastern North America are shown in Figures 7.A-18 and 7.A-19. It is intended that these curves represent soils in the general range of gravelly sands to low plasticity silty or sandy clays. They should not be applied to either very gravelly or very clayey deposits. As is the case with the curves presented in Figures 7.A-10 and 7.A-11, they do not account for the effects of degradation and degeneration of stress-strain loops, so that for materials which develop any excess pore pressures under the levels of loading in question, the damping ratios at larger strains likely err in the conservative direction.

While generic curves of the kind given in Figures 7.A-10 through 7.A-19 are suitable for preliminary studies, soil-specific modulus reduction and damping curves and their variation with depth should be obtained for final site-specific studies of site response. In this connection there are two potentially important factors which remain unresolved: (1) the effect of overconsolidation; and (2) the effect of sample disturbance on measured modulus reduction and damping curves. Vucetic and Dobry (1991), for instance, report that overconsolidation is not a significant factor but it is possible that the effects of overconsolidation are masked by: (1) sample disturbance in testing natural soils, and (2) the fact that many laboratory tests reported as being overconsolidated have been conducted using isotropic overconsolidation rather than overconsolidation with no lateral strain, such as occurs in the field. Since the ratio $G_{\max}/S_u$ for clayey soils decreases with increasing OCR, the reference strain, which is proportional to the inverse of $G_{\max}/S_u$, must increase, and therefore one would expect that overconsolidated soils might show increasingly linear behavior with increasing OCR. It is generally believed that the effect of sample disturbance is to move the modulus reduction and damping curves in the opposite direction, that is, to make them more nonlinear (for example, Anderson and Stokoe, 1978), so that OCR and sample disturbance would act in opposite directions. However, it can also be argued that since sample disturbance might be expected to reduce $G_{\max}$ more than $\tau_{\max}$, that disturbance should increase the reference strain and lead to more elastic behavior (Larkin and Taylor, 1982). It is not possible to resolve these issues with currently available data but this might be done in a carefully planned laboratory test program in which the effects of overconsolidation, stress release and mechanical disturbance are simulated under controlled conditions.

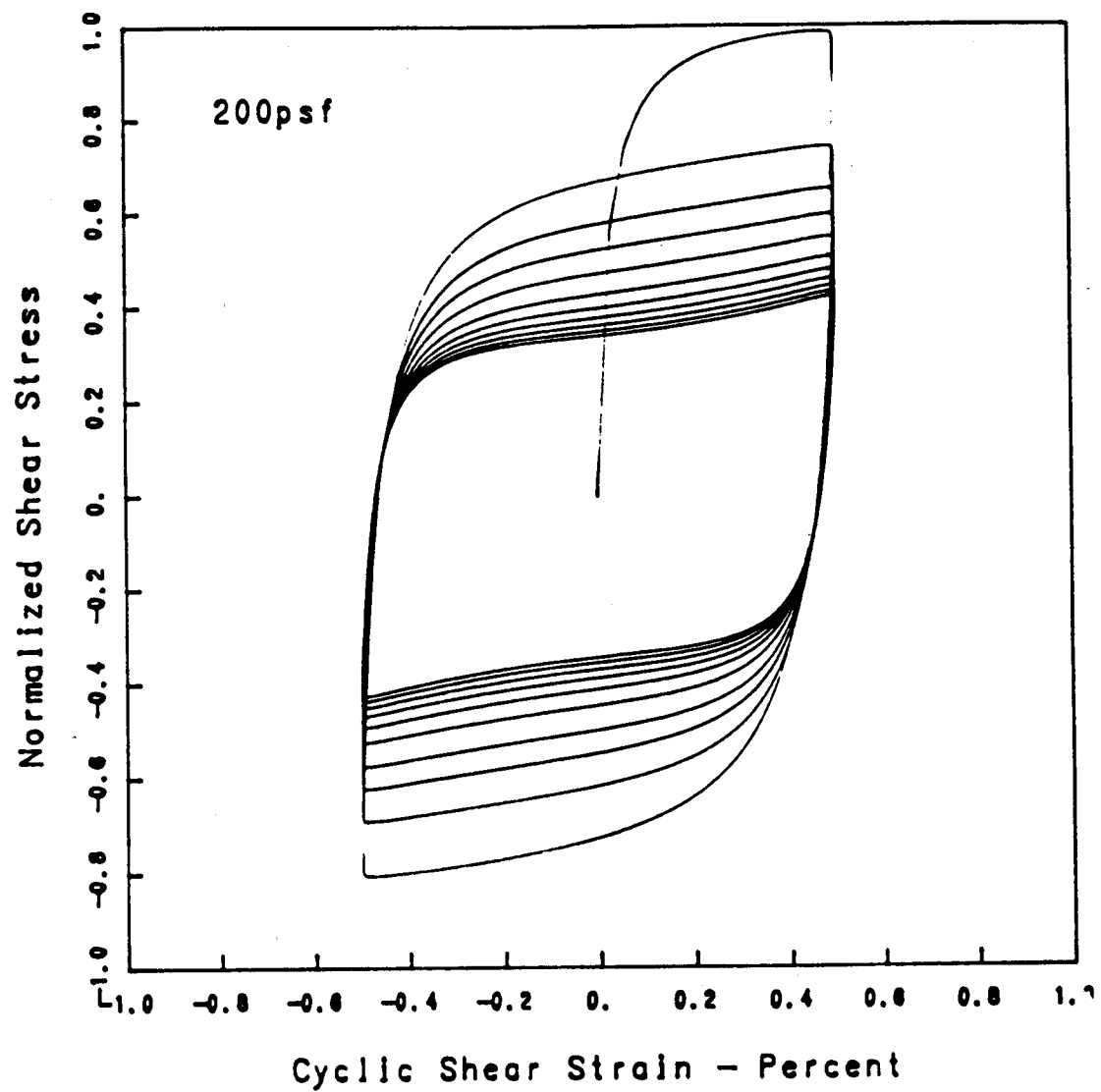


Figure 7.A-9a
Typical Stress-Strain Loops for Sand

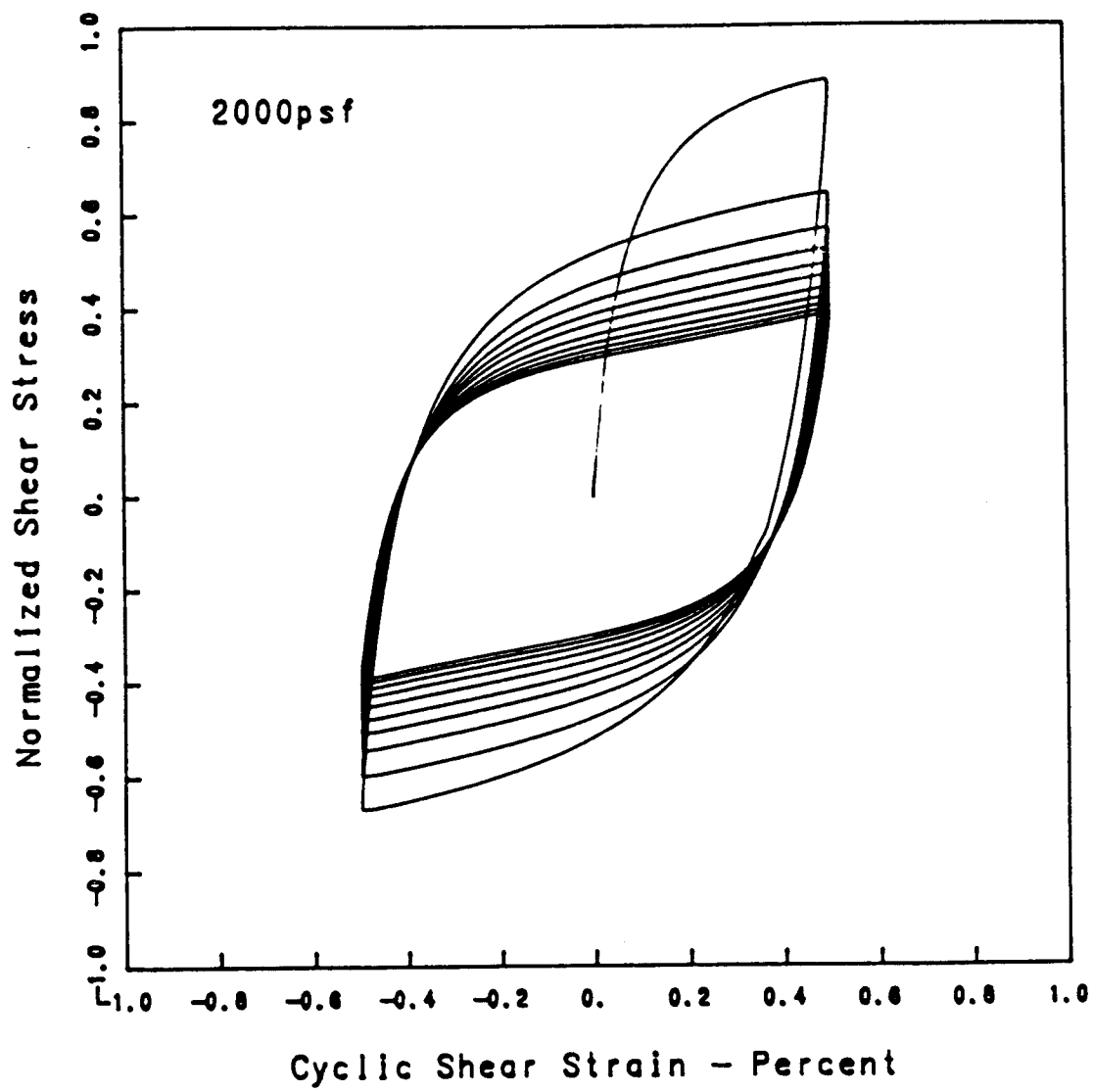


Figure 7.A-9b
Typical Stress-Strain Loops for Sand

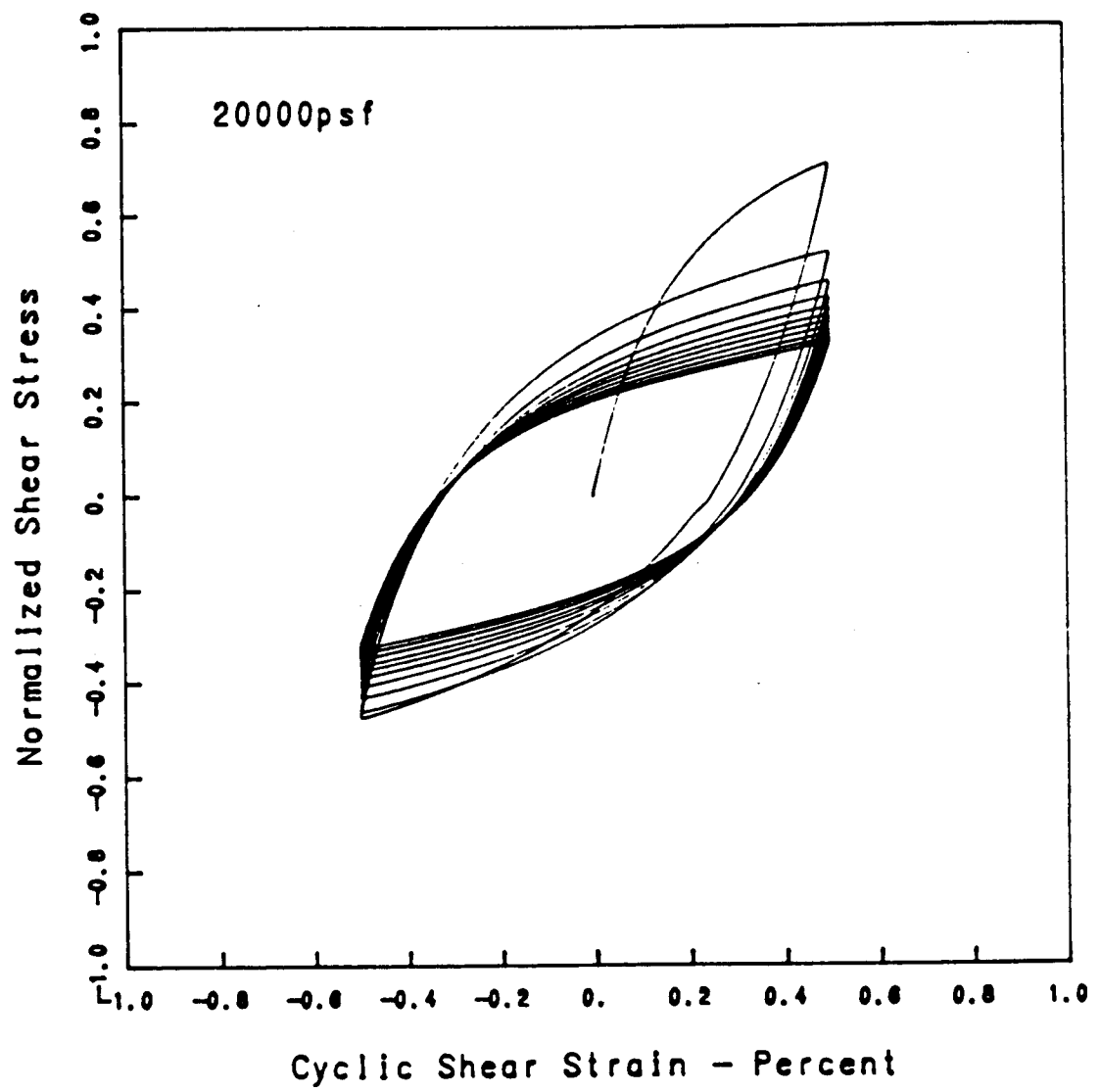


Figure 7.A-9c
Typical Stress-Strain Loops for Sand

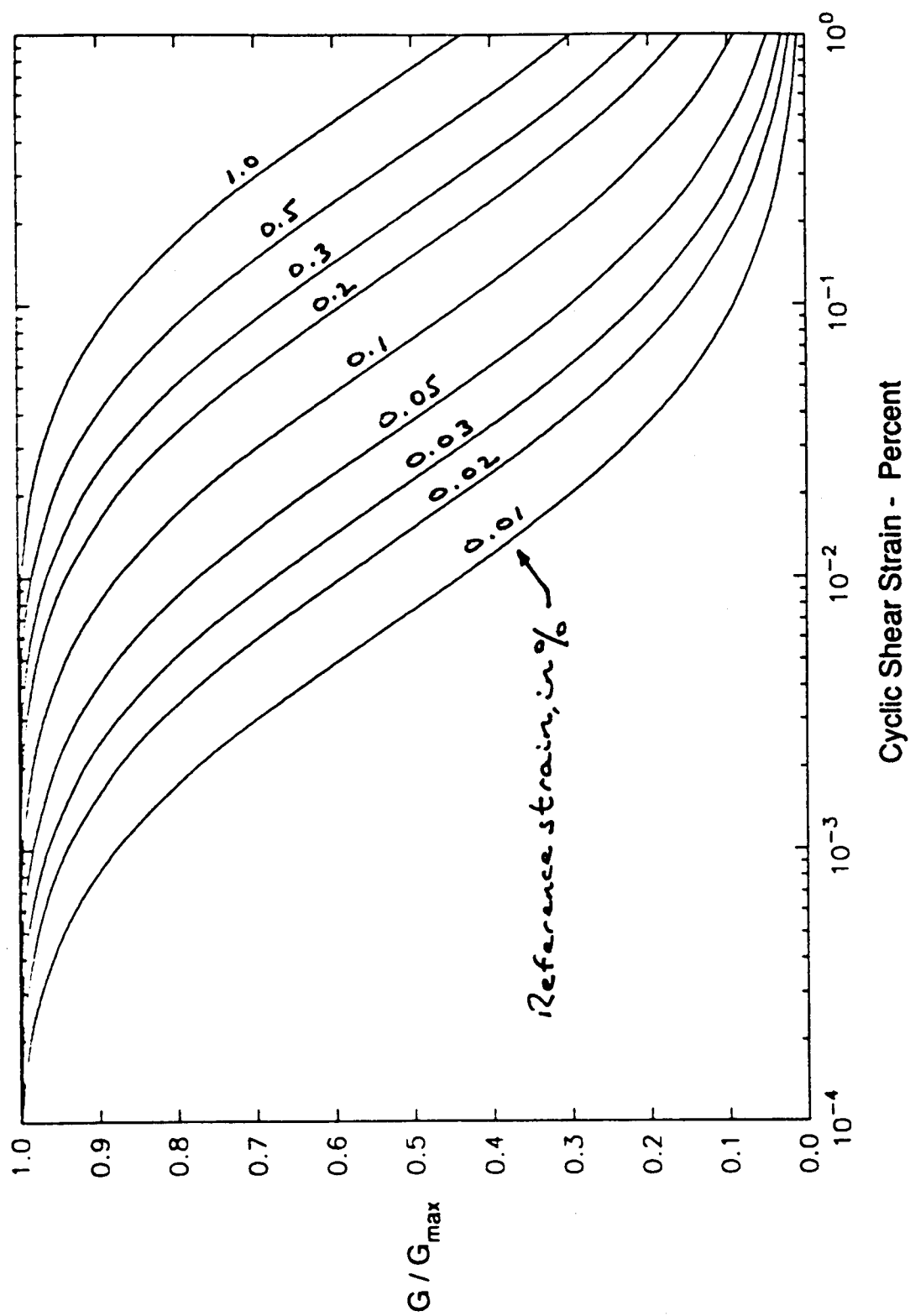


Figure 7.A-10
Modulus Reduction Curves as a Function of Reference Strain

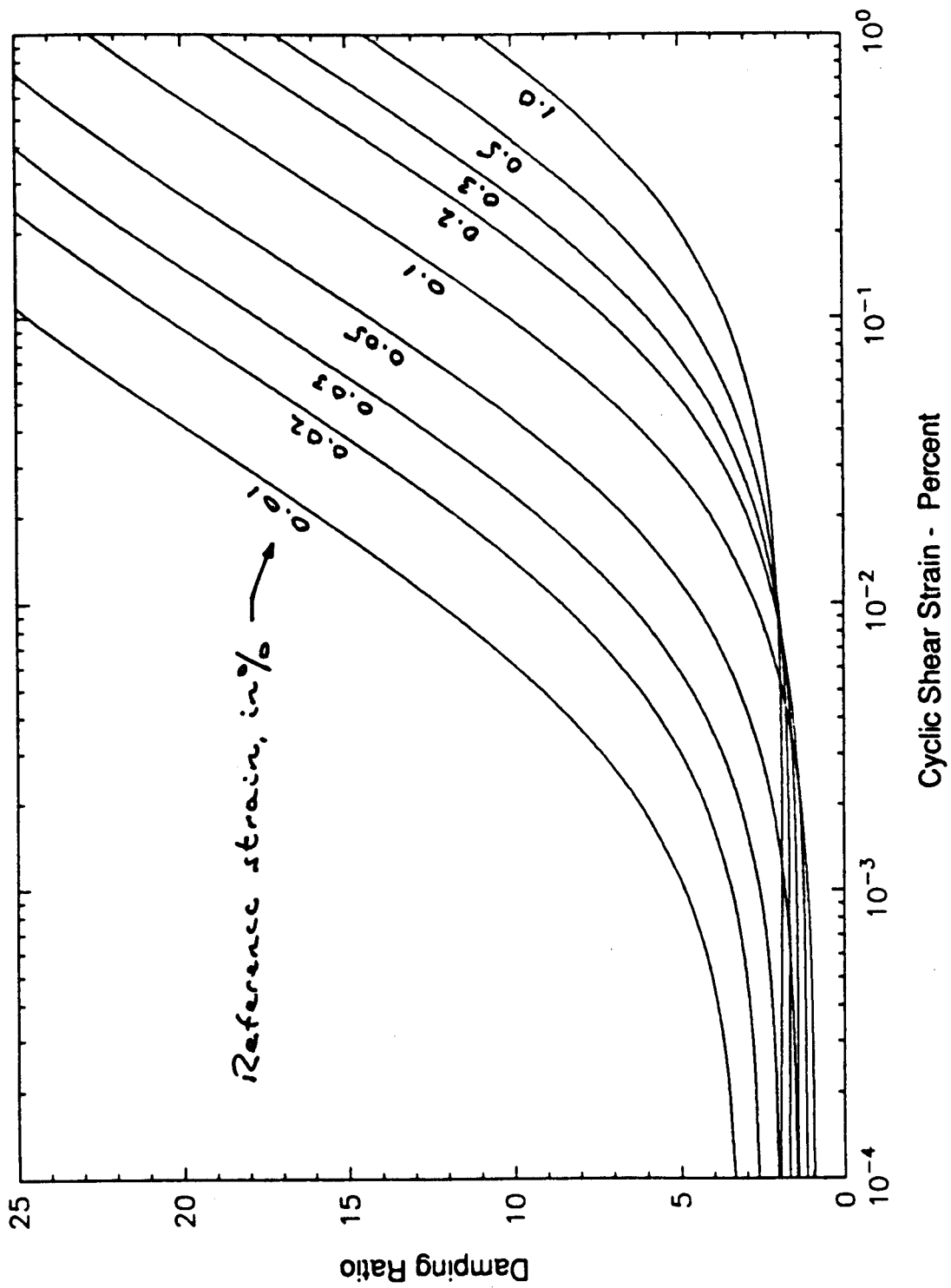


Figure 7.A-11
Damping Curves as a Function of Reference Strain

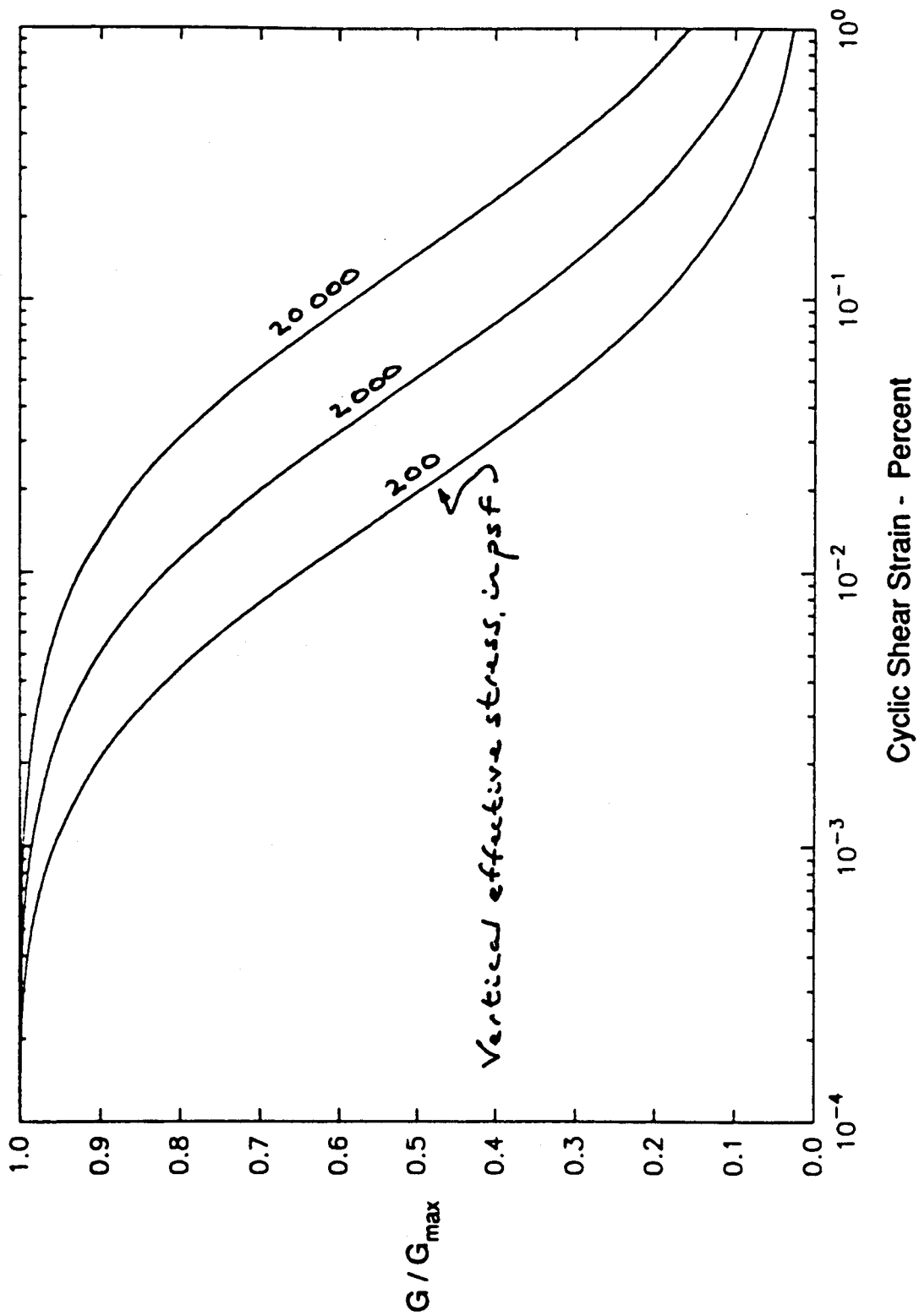


Figure 7.A-12
Modulus Reduction Curves for Dry Sands

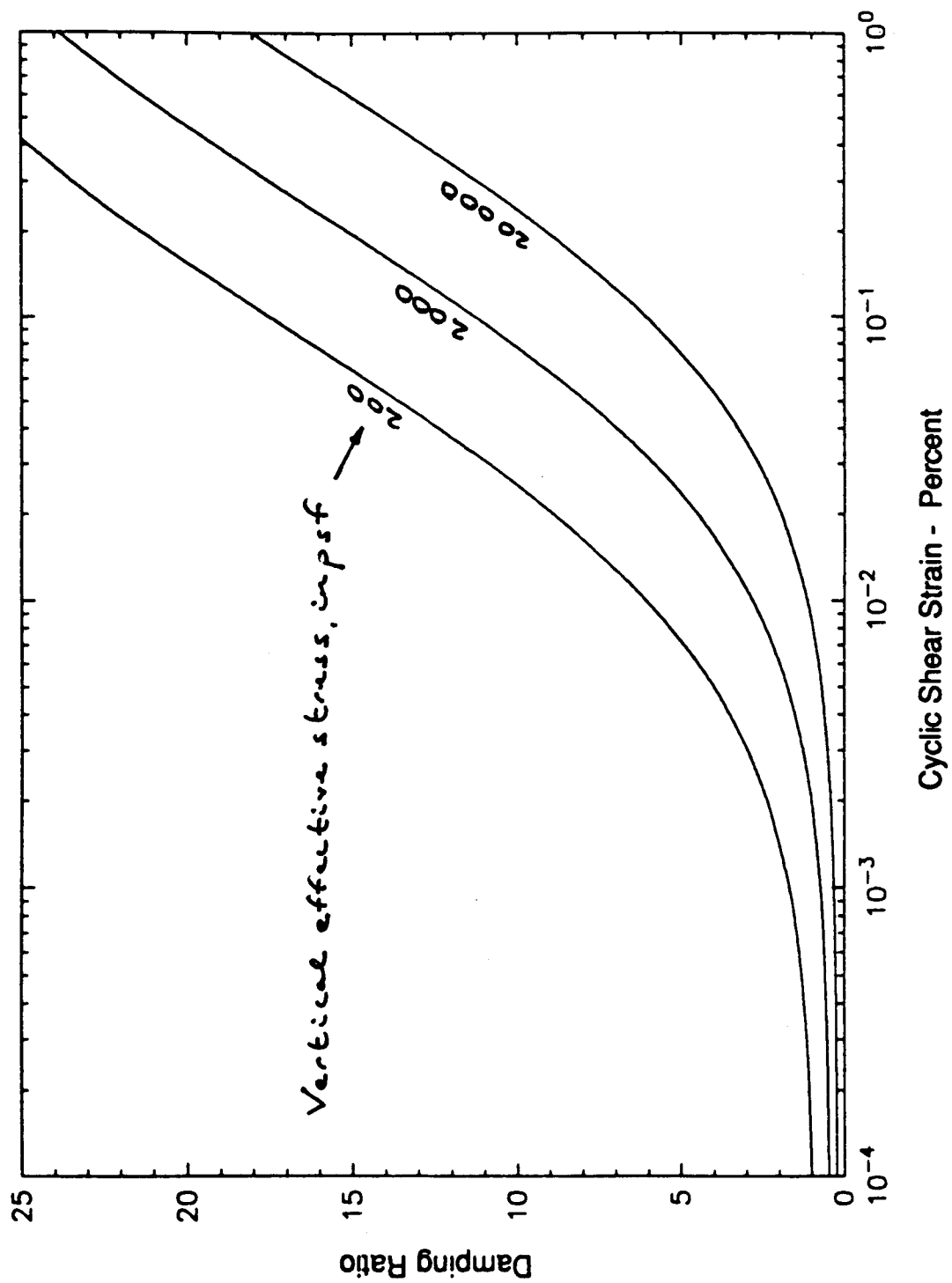


Figure 7.A-13
Damping Curves for Dry Sands

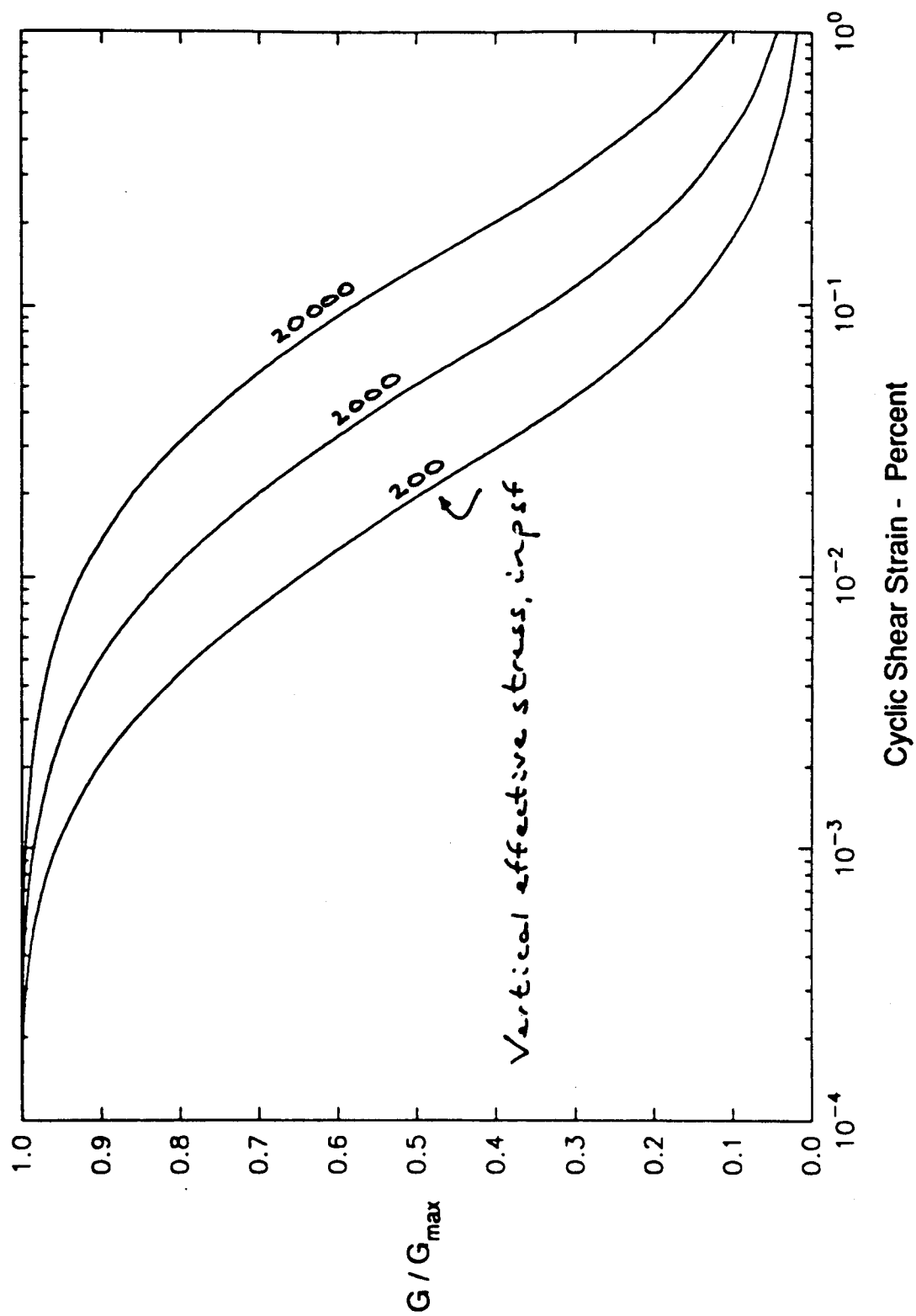


Figure 7.A-14
Modulus Reduction Curves for Saturated Sands

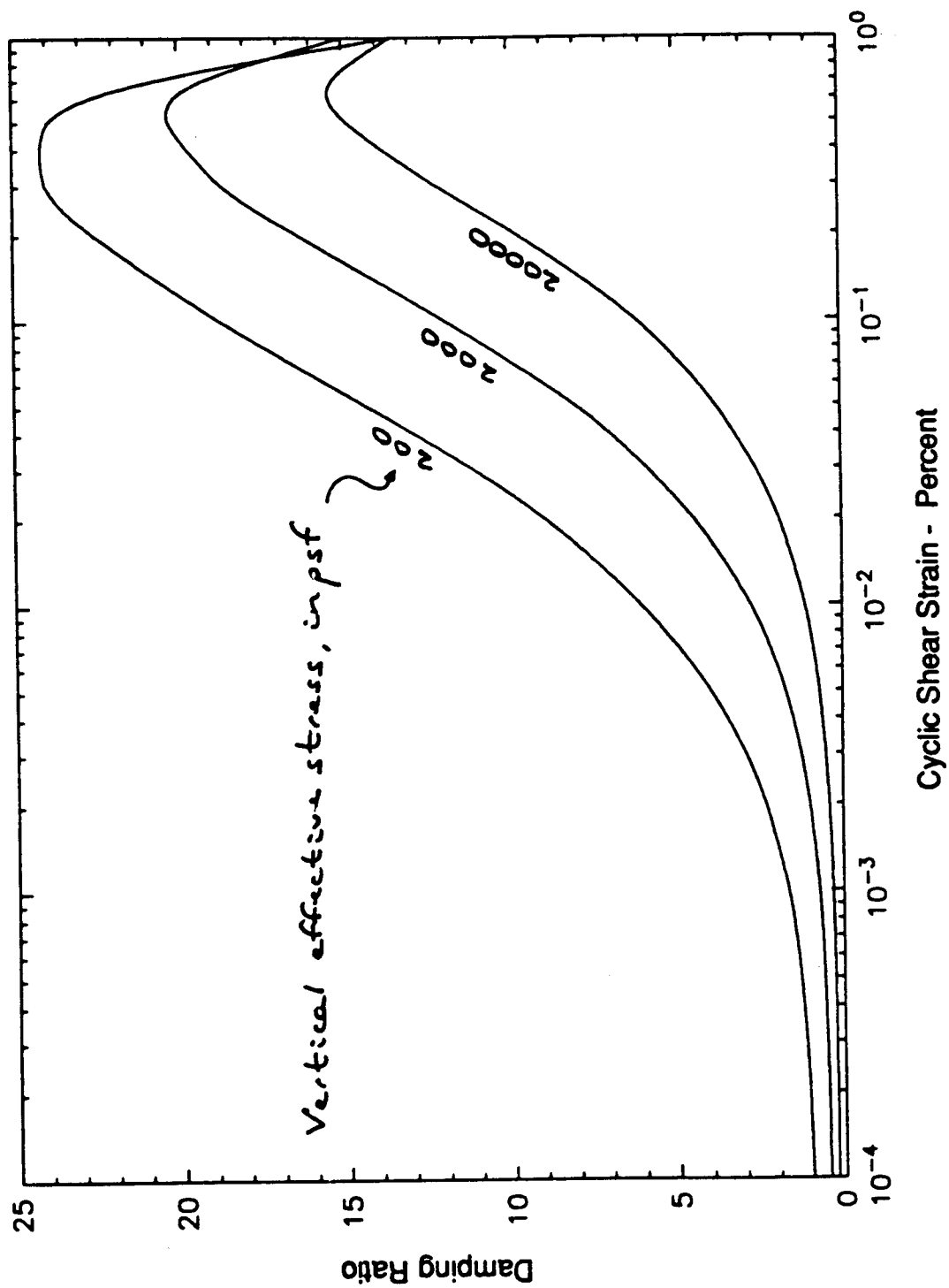


Figure 7.A-15
Damping Curves for Saturated Sands

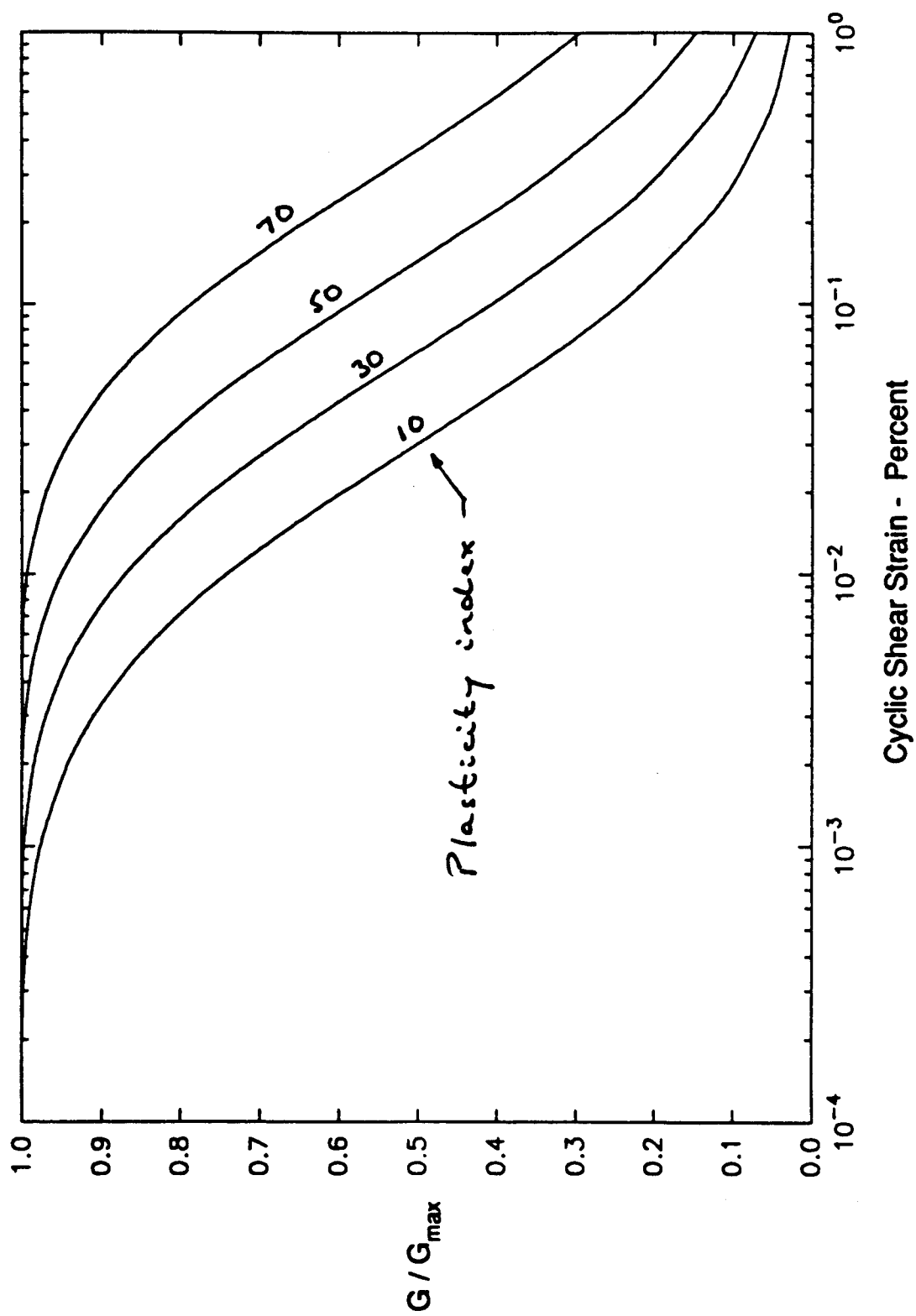


Figure 7.A-16
Modulus Reduction Curves for Clays

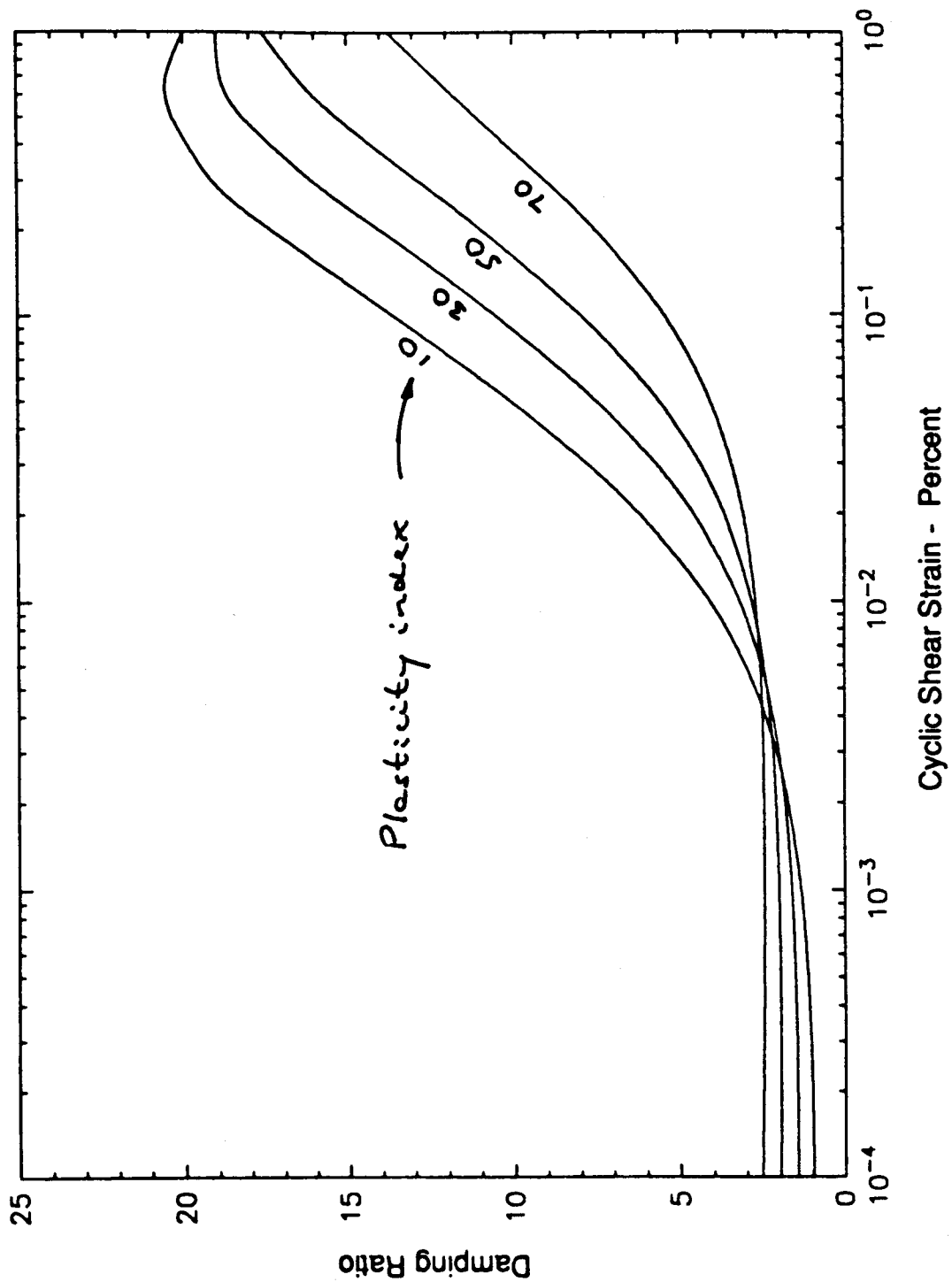


Figure 7.A-17
Damping Curves for Clays

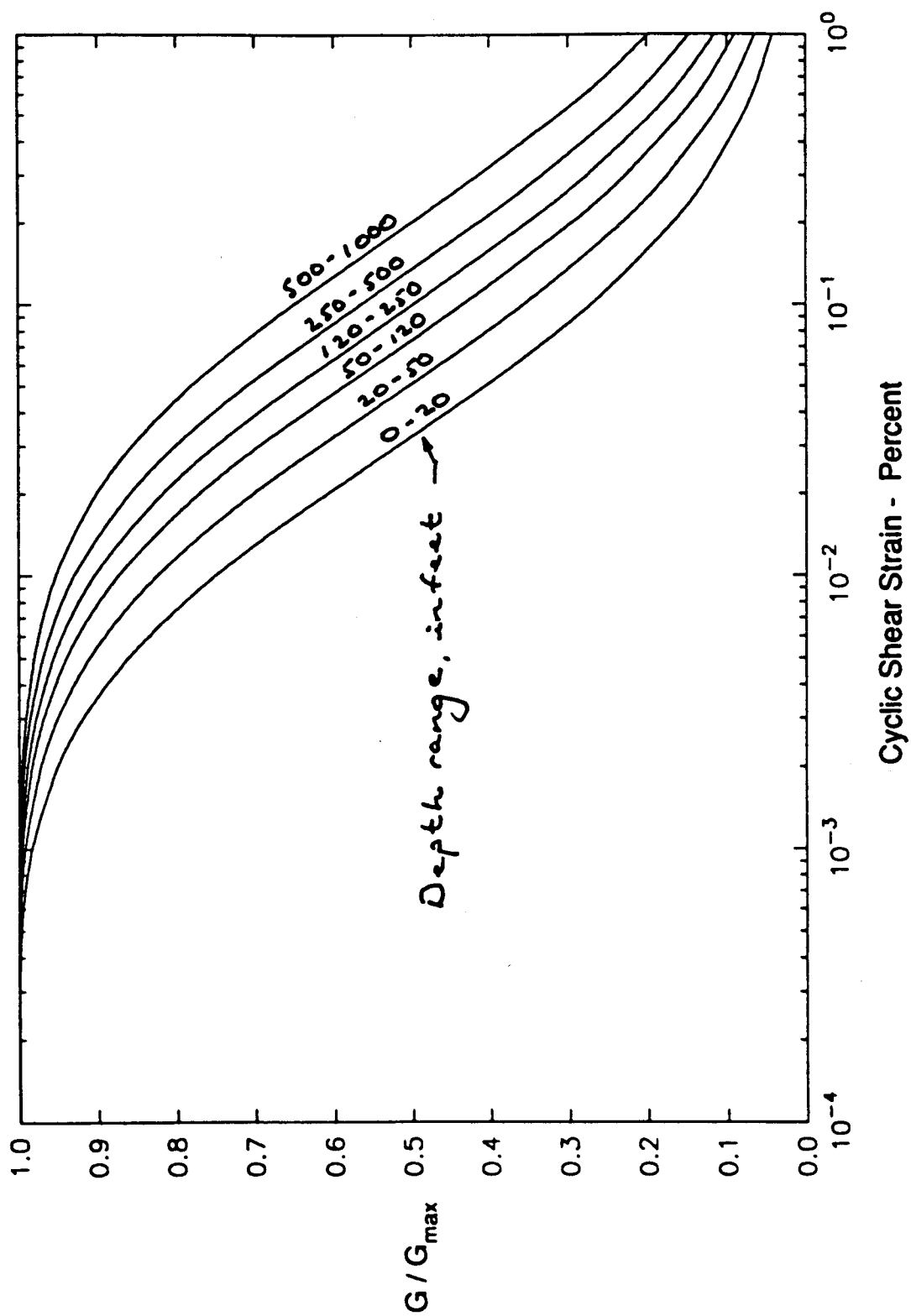
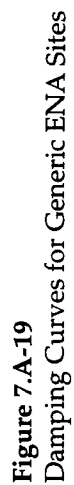


Figure 7.A-18
Modulus Reduction Curves for Generic ENA Sites



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